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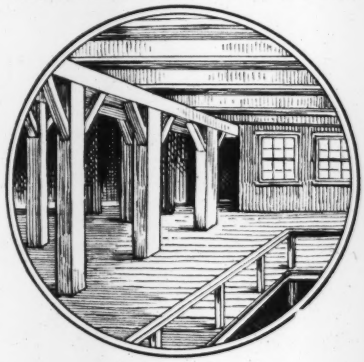
KYANIZED SPRUCE

WIDENS THE FIELD OF WOOD CONSTRUCTION



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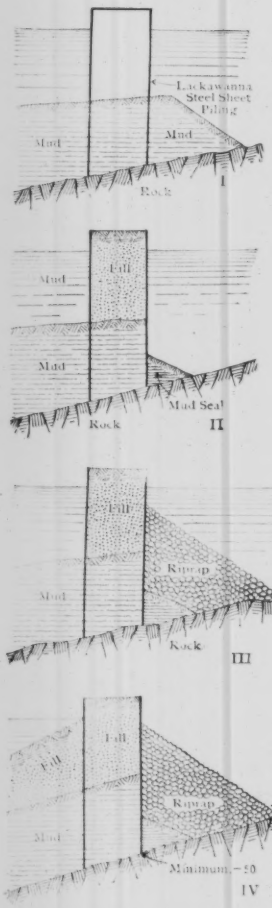
Steps in the construction of the Lackawanna Steel Sheet Piling Cofferdam for 46th Street Piers, New York City



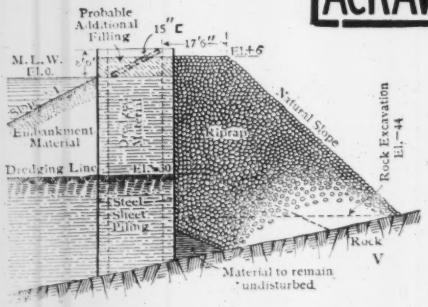
The above view shows the river or deep water side of the cofferdam when all the Lackawanna Steel Sheet Piling had been driven and before the anchor piles in the cofferdam had been removed.

The progressive steps in building the dam after the installation of the steel sheet piling pockets are shown in the sketches. The steel sheet piles were driven through an overlying 20 feet or more of mud and silt as a bottom support and seal on top of bed rock as shown in Step 1.

Step II was to tub out the mud lying on the shore side of the cofferdams and into the cell of the dam, leaving a seal at the inner toe of the dam. Step III was to dump part of the riprap against the shore side of the dam, and Step IV to place the fill of mud up against the river side. Step V includes completion of placing of riprap on the shore side and pumping out the dam, and shows a typical section of the finished dam. Examination of these diagrams will show that at various times during the process, the dam would be in rather precarious state of equilibrium if the processes indicated on the steps were continued entirely across the dam. For instance, in Step III, if the riprap were placed *entirely* before the fill was placed on the river side there would be a pronounced tendency to turn over toward the river.

In order to avoid such possibility the filling and excavating on either side of the cofferdam were carried out with extreme caution in alternate sections along the dam, so that no great length of the dam was subjected to excessive pressures from either side at any one time.

Additional construction details will be given in next week's advertisement.



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Engineering News

VOLUME 75 MARCH 9, 1916 NUMBER 10

Electric Dredge Used on River Improvement in Washington

By J. H. Walter*

A 20-in. electric suction dredge, the "Puyallup No. 1," is now in service in Washington, engaged in removing 3,000,000 cu.yd. of subaqueous material. This work is a part of the Inter-County River Improvement, a joint project undertaken by King and Pierce Counties, Washington, for the purpose of relieving the flood damage of the White-Stuck and Puyallup Rivers. Widening, deepening and straightening the channel require the removal of 4,620,000 cu.yd. A contract has been let for straightening 8 mi. of the channel and the removal of 3,000,000 cu.yd., based upon the counties' furnishing a 20-in. suction dredge.

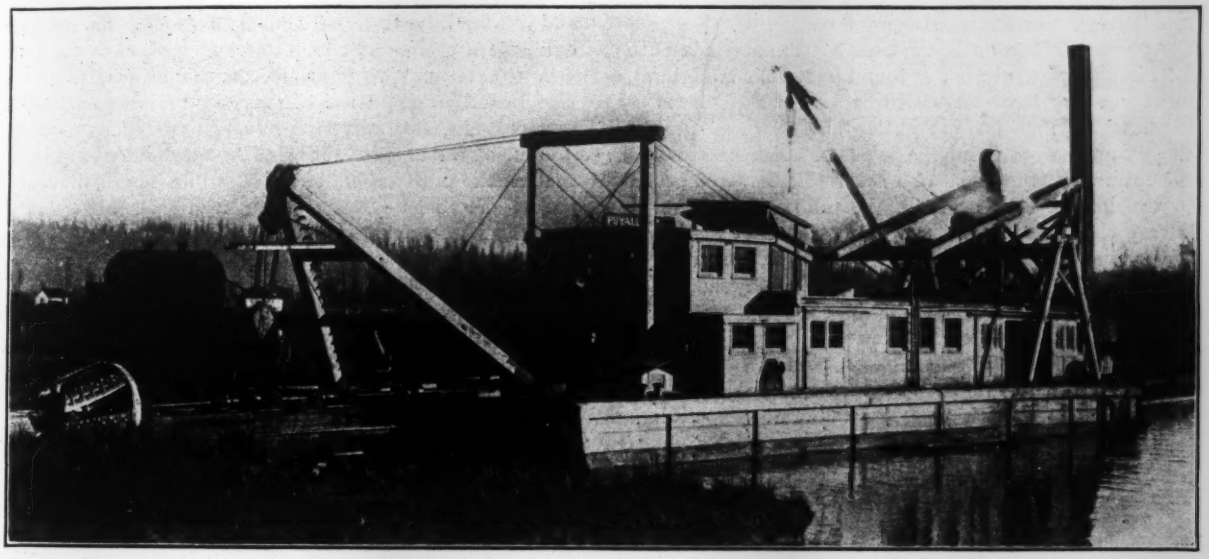


FIG. 1. ELECTRIC DREDGE "PUYALLUP NO. 1," BUILT FOR KING AND PIERCE COUNTIES, WASHINGTON

The dredge was designed by the writer for the counties and built under contract, at a cost of $70,000. It is now being operated at a low unit price for yardage, but carries with it an agreement to purchase the dredge at 75% of its original cost, if the counties so elect, at the end of the contract.

This dredge is electrically driven, but has a steam auxiliary on the line shaft of the winding machinery. It has a timber hull 9 ft. 6 in. by 40 ft. by 115 ft., in which there are 172,000 ft. b.m. of lumber and over 27,000 lb. of iron and steel. The dredge draws 3 ft. 9 in., showing a displacement of about 500 tons.

The bottom is made of 10x12-in. timbers, all 40 ft. long. The longitudinal bulkheads, seven in number, are of 10x18-in. timbers, five strakes each; deck beams and stanchions are of 10x10-in.; the crossbraces are of 4x12-in.; and the deck is of 4x6-in., while the cross-bulkheads are of 8x18-in. timbers. The hold is divided into 10 fairly equal water-tight compartments. A 9-ft. 9¾-in. by 24-ft. well in the forward end takes the ladder trunnions, thus minimizing the overhang of the A-frame supporting the ladder hanger. The main deck house provides no quarters for crew, but simply houses the machinery, with a roomy pilot house set in the upper half in the forward end. In the pilot house are controls for the complete operation of the dredge, with the exception of the priming of the main pump and the operation of the bilge pump.

The ladder is built up of four 24-in. 100-lb. I-beams, two on each side, with heavy cast-steel strong-backs and lattice bracing. It is 50 ft. long from the center of the trunnions to the back ring of the cutter head and can be operated in any position from horizontal to vertical. The cast-iron suction pipe, 20 in. in inside diameter, passes through the starboard trunnion, while the cutter drive is transmitted through a countershaft in the port trunnion. Both of these passages through the trunnions have good and sufficient clearance with the sides of the

*Assistant Engineer, Inter-County River Improvement, Tacoma, Wash.

FIG. 2. TWENTY-INCH PUMPING UNIT OF ELECTRIC SUCTION DREDGE

trunnions. The original cutter was designed for very soft digging, was 8 ft. in diameter, of the helmet type, and was driven by a 100-hp. 600-r.p.m. reversible and variable-speed motor. By making the bull-wheel 86 in. in diameter only three reductions were found necessary to obtain the proper speed for the cutter.

At present there is being used a straight-blade cutter, 6 ft. in diameter, which was found advisable on encountering some very hard clay. No babbitt is used in the two forward bearings on the cutter shaft, where this gets the extreme wear from the sand and silt stirred up by the cutter. But in its stead a removable cast-iron bearing, in two halves, is inserted in the pillow block, affording a very cheap and quick renewal. The shortest bend of the suction pipe has a 5-ft. 1-in. radius, and there are only three bends on the dredge in both suction and discharge pipes.

The main pump is of the "convolute" type, as designed and built by the Bucyrus Co., of South Milwaukee. It has a steel shell and runner, with renewable front and back liners of soft-steel plate and a medium-steel nose ring—also renewable. The pump is direct connected through a friction relief coupling, of the same company's latest design, to an 800-hp. 360-r.p.m. variable-speed motor. The unit as a whole is assembled on a bed 7 ft. 6 in. by 26 ft., built up of six parallel 12-in. I-beams and ½-in. steel plates top and bottom. There are enough intercostals in this bed to make it a most rigid foundation, and it takes its seat directly on the five center longitudinal bulkheads.

A 4-in. steam ejector primes the pump in 35 to 45 sec. The back flow from the discharge pipe is taken care of by a cast-steel flap valve.

Four of the units of the winding machinery have interchangeable pin-friction clutches, while the fifth unit—the ladder hoist—has a band friction. All five are assembled on one line shaft at the forward end of the main deck house, the line shaft extending to outside gypsies. This line shaft is driven through one center shaft by a 75-hp. 600-r.p.m. reversible and variable-speed motor. In addition there is an auxiliary 10x12-in. double steam engine, connected to the line shaft with a jaw clutch. The controls for the winding machinery are arranged in the pilot house with hand levers for the friction and with foot levers on the brake bands. This arrangement gives the leverman quicker action than the

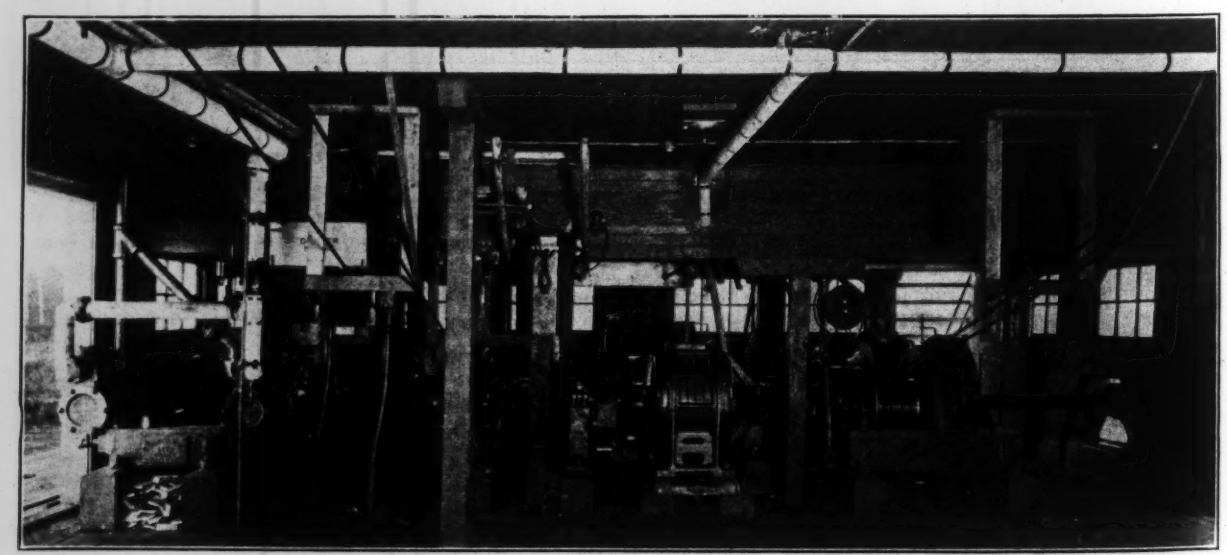


FIG. 3. WINDING MACHINERY OF ELECTRIC SUCTION DREDGE "PUYALLUP NO. 1"

hand levers on the brakes. The machinery was manufactured by the Washington Iron Works, of Seattle. A view of the winding machinery is given in Fig. 3.

Each one of the 10 water-tight compartments has a 6-in. suction and a gate valve governing it, from a 12x7x12-in. duplex steam pump. It will deliver 400 gal. per min. and, having also an outboard suction, is connected up to four 2½-in. hydrants for fire protection. This pump also supplies the circulation for the water jackets of the thrust bearings and stuffing-box on the main pump. To supply steam for this duplex pump and also the auxiliary engine on the winding machinery, there is on the stern of the dredge a horizontal boiler having 582 sq.ft. of effective heating surface. The necessary steam lines are also provided for portable siphons all over the hull and for heat in the pilot house.

Drum-type controllers were used for the two smaller motors, while the main pump motor is controlled through a master control and contactor panels. The resistance grids are housed in a well-ventilated addition on the back of the pilot house. All the electrical equipment was made by the General Electric Co.

In addition to the dredge proper there were furnished twelve 12x24-ft. pontoons, 500 ft. of standard pontoon pipe, 3,000 ft. of standard levee pipe, gates, elbows, sleeves, etc. There was supplied also a 4-ft. 6-in. by 26-ft. by 48-ft. derrick scow. On this scow is a 10¼x12-in. three-drum hoisting engine and a stiff-leg derrick with a 40-ft. boom. This derrick scow was designed for general use, but most particularly for clearing the river of snags and drift.

The Dredge Tested Before Service

Before delivering the dredge to the contractor for operation, a thorough test run was made. The discharge was extended 3,000 ft. and elevated at the end to 25 ft. above the surface of the water at the dredge. The velocities were obtained by putting 6-in. wooden balls, partly loaded, into the suction pipe at the cutter head and also by emptying gallon jugs of potassium permanganate. In all, 18 of these trials were made, with fairly good results. The vacuum, pressure and power consumptions were taken as averages over a period of 19 hr. of steady pumping. During this time only clear water was pumped, to obtain more uniform readings than can be obtained while digging material.

The following results were obtained:

Average pressure	= 43 lb. =	98.9 ft. head
Average vacuum	= 19 lb. =	21.5 ft. head
Total pressure		= 120.4 ft. head

Velocity as measured = 15.3 ft. per sec. Therefore the output was $\frac{15.3 \times 2.18 \times 62.5 \times 120.4}{550} = 456.3$ hp.

The power consumed was 505.8 kw.-hr. = 678 hp. input. The efficiency of the unit was $\frac{456.3}{678} = 67.3\%$.

The efficiency of the pump proved to be $\frac{67.3}{90} = 75\%$, the motor efficiency being 90%.

Tweeden & Marsh, of Tacoma, were the general contractors for the construction of the dredge. Puyallup Dredging Co., of Seattle, is the contractor for the dredging. W. J. Roberts is chief engineer of the project.

Interior Wall Finish in Panama Canal Hospital Buildings

The sulphate of zinc treatment for concrete walls to be afterward painted has been adopted for the interior wall finish of the permanent hospital building at the Panama Canal. The report of Geo. M. Wells, Resident Engineer of the Building Division, describes these buildings and their interior wall finish as follows:

A great deal of study was given to the design of these buildings in an effort to decide upon that form of composite construction that would make for economy and yet meet all the rigid requirements demanded by the health department as regards character of interior and exterior finish. It was finally decided to make the veranda and main-building walls of poured reinforced concrete, while the floors with a clear span from wall to wall across the building were to be constructed of reinforced concrete overlaid with terrazzo mosaic tile or some composition flooring for finish. Intermediate walls 6 in. or over in thickness are also to be built in reinforced concrete. Walls less than 6 in. thick are to be laid up of hollow cement blocks plastered with two-coat cement and sand stucco troweled to a smooth dead surface. The concrete walls are to be rubbed and filled to a similar finish.

The health department demanded that walls should be both smooth and of sufficient hardness to withstand chipping at corners, cracking or denting, and further that the treatment of the wall finish should be such that the wall could be washed with antiseptic solutions if necessary, without in any way injuring the finish.

It is proposed to meet these requirements with the cement walls as described, treated with chemical and painted with special paints as follows: The walls will be first given two coats of a wash of sulphate of zinc, mixed with equal parts by weight of zinc and water. The effect of this wash is to neutralize or destroy the alkalinity of the hydrated lime which comes to the surface of the concrete wall as calcium hydroxide and changes the latter into a mixture of calcium sulphate and zinc oxide. This gives a surface that has no leaching effect on linseed oil. Two coats of flat-finish oil-mixed paint of any color desired will then be applied. On this will be placed as a final finish a third coat of enamel washable wall paint.

It can be shown that a concrete wall treated in this manner will cost far less per square than the usual three-coat hard-finish white plaster sized and tinted to the color desired. The difference in cost between the two methods is estimated to be approximately $12 per 100 sq.ft. in favor of the painted wall with the additional advantage gained of a harder and more durable surface.

The just described sulphate of zinc treatment for concrete walls that are to be painted was discussed by Charles MacNichol in a paper read at the annual meeting of the American Society for Testing Materials in 1910. Mr. MacNichol recommended that the solution of zinc sulphate and water be applied with an ordinary bristle brush after the cement is dry, and that after application 48 to 72 hr. be allowed for the wall to dry before applying paint. Mr. MacNichol's paper also contained a statement by Dr. Allerton F. Cushman describing the chemical action of the sulphate of zinc in the same manner as reported by Mr. Wells and expressing approval of the treatment.

Cost of Highway Surveys—According to data recently compiled by the State Engineer of Oklahoma, W. P. Danford, the cost of making surveys for state roads has varied from $2.95 to $33.33 per mi., the average for 1,370 mi. being $10.05. The work was done by county engineers, who receive $5 per day and expenses. The expenses include the pay of two assistants at $2 per day and transportation. The average of 46 counties was $10 per mi., and leaving out the few high ones, the average cost of the remaining 38 counties was $6.70 per mi. This includes both alignment and level lines and the drafting of a plan and profile. The alignment of Oklahoma roads is not difficult to survey, for they invariably follow section lines, north and south and east and west. The county commissioners are reluctant to change the alignment for any other reason than sheer impassability, and this constitutes one of the state's highway improvement problems.

Concrete Strength Not Result of Aggregate Strength

By Thomas Hendrick*

To one who has had recourse to the literature upon aggregates in concrete it is evident that first-hand information in regard to the relative efficiency of different kinds of stone, when employed as coarse aggregate, is lacking, and the information that can be found is not conclusive. The data in handbooks are based upon the fact that stones ranging from very hard to very soft will, when introduced into concrete as aggregate, give a resultant that varies in strength directly as the strength of the stones themselves. Such a conclusion seems to be taken for granted. Standard texts, moreover, assume from the fact that experiments on the same aggregate give the result that the "densest stones make the strongest concrete" that such a fact is true for all stones, although differing in kind. From other data on tests of concrete made from traprock alone is based the conclusion that the strength of concrete is proportionate to the shearing value of the stone.

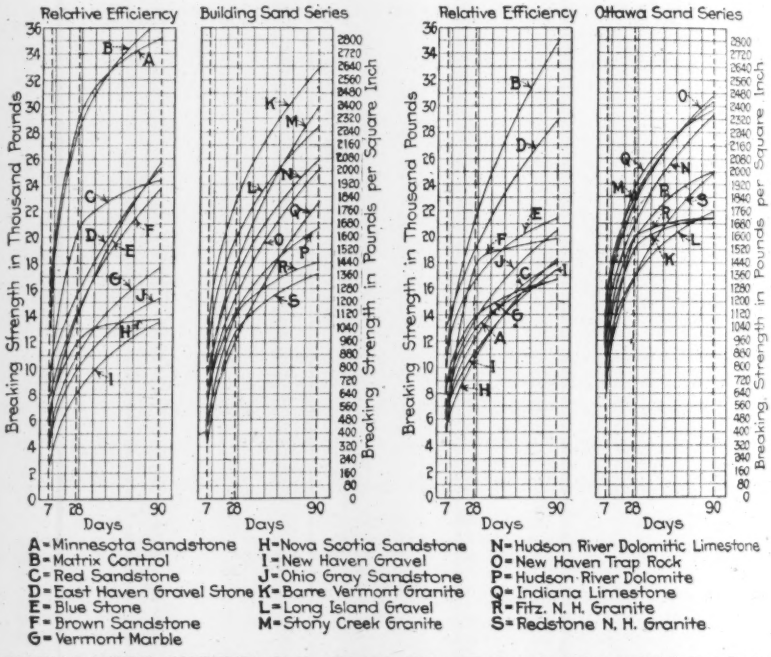
In this article are given the results of experiments performed at the Cement Testing Laboratory of the Sheffield Scientific School, of Yale University, showing the relative efficiency of 16 kinds of broken stone and two kinds of gravel when used as aggregate; and from these results the importance which should be placed upon the factor of adhesion in the consideration of a stone as an aggregate is developed.

An elaborate description of the stone and gravel experimented upon is out of place here, but a brief mention of the commercial names and essential characteristics is of value. The materials used were as follows: Five sandstones—Connecticut brown, Nova Scotia drab, Massachusetts red, Minnesota Kettle River pink and Ohio gray; Vermont marble; Indiana limestone; two gravels—New Haven and Long Island; Hudson River dolomite; Hudson River dolomitic limestone; four granites—Fitzwilliams (N. H.), Barre (Vt.), Redstone (N. H.) and Stony Creek (Conn.); and New Haven traprock.

Of the sandstones the most easily crushed was the Minnesota. Its grains were clean and granular, as compared with the earthy texture of the other sandstones. Of the granites Redstone and Stony Creek were similar in reddish color and in resisting crushing, and Barre resembled Fitzwilliams in its physical characteristics. Indiana limestone in texture was something like Ohio gray sandstone, its cementitious material being of a somewhat earthy substance. The grains of East Haven gravel stone were sharp and glass-like. The traprock, dolomite, dolomitic limestone and bluestone were of such structure that it was necessary to employ the usual acid test to tell them apart. Vermont marble reduced itself, when struck with a hammer, to a substance that looked like salt. The sea-washed hard Long Island gravel contrasted with the clayey impure New Haven gravel.

Actual figures for crushing strength of seven of these stones, taken from Edwin C. Eckel's book on "Building Stones and Clays; Their Origin, Characters and Examination," are as follows: Strength in pounds per square inch, Barre (Vt.) granite, 17,182; New Haven trap, 9,500; Connecticut brown standstone, 11,696; Massachusetts sandstone, 11,517; Indiana limestone, 5,635; and Vermont marble, 8,670. These figures show the relative strength of the raw stone, a general knowledge of which is all that is essential to the present purpose.

It may be mentioned, however, that tests of stone in the raw show considerable difference from tests of the stone as it is used in the aggregate. Here it is used not in a solid mass, but in small particles, the natural cementitious material having been replaced by the matrix used; and tests of raw stone cannot be taken as an index of the way in which a particular stone will act as an aggregate.



COMPARATIVE COMPRESSIVE STRENGTHS OF CONCRETE WITH CERTAIN AGGREGATES AND OTTAWA AND BUILDING SANDS

By making compression tests on cylinders (size 4x8 in.) of concrete, proportioned 1:2:4 by volume reduced to weight, the aggregate alone being varied, a comparative value of the different stones as aggregate was determined. The stones were made of the same size by passing through a 1-in. and retaining on a ½-in. sieve. To secure uniformity, the same kind of sand and cement was used throughout, two series being used—building-sand and Ottawa sand series. Standard methods of mixing and proportioning were employed. The comparison is based upon tests of 7, 28 and 90 days, for which curves are shown herewith.

The results obtained are given both in actual breaking strength and in pounds per square inch. Generally speaking, the tests of 28 and 90 days emphasize the results of 7 days. Tests for the Ottawa sand series run high

*43 Iden Ave., Pelham Manor, N. Y.

which tests for the same stones in the building sand series run low. Special attention should be given to the curves of Minnesota sandstone in the building-sand series and to Indiana limestone in the Ottawa sand series, also to that of the matrix control in both series. The Minnesota sandstone and the Indiana limestone were the two softest stones used; yet the testing values are higher in the case of the Minnesota and about as high in the case of the Indiana for their respective series. The matrix control maintains throughout comparatively high results.

What the Experiments Show

From the outcome of the tests it cannot be asserted that shearing strength is the criterion of the fitness of an aggregate to give strong concrete. The other forces resisting compression—the shearing strength of the matrix and the interlocking factor, adhesion—must be considered with it, the three forming a unit of resistance. The difference of breaking values for similar stones in the two series is explained not by the effect of changing the shearing strength of the aggregate, but by the change in the factor of adhesion when a new aggregate is introduced. In the building-sand series the matrix, in the case of the Minnesota sandstone, has a greater adhesive force (by reason of the superior testing values) than it has in the case of the other stones, because with this stone as aggregate the matrix makes an entirely homogeneous mass, or a more nearly monolithic structure; and similarly in the case of the Indiana limestone in the Ottawa sand series the comparatively high testing value is due to adhesion of the matrix and aggregate. In selecting these two examples, moreover, the exceptions are not taken, because of the four granites used, Stony Creek and Redstone, nearly equally strong in resisting shear when raw, failed to show the same comparative results when used as aggregate; and in the same way Barre granite exceeded the Fitzwilliams granite when used in the concrete, although practically equal in shearing strength in the raw stone.

It is maintained by some that it does not make any difference what stone is used as aggregate, so long as its shearing strength is far in excess of that which the matrix will stand; that the matrix will fail anyway; and that if the adhesion is strong enough, the ultimate strength of the concrete will approach, but in no case can it exceed, the value of the shearing strength of the stone. Yet the specimens of pure matrix failed at a value far in excess of the value for which specimens of granite aggregate fail—a fact which nullifies the theory that the strength of the matrix determines the ultimate strength of the concrete and which not only emphasizes the importance of adhesion, but seeks for its very explanation in the power (high or low) of the matrix to interlock with the aggregate, whether the latter is granite, trap, sandstone or limestone.

Compression Depends on Adhesion

It is hard to see any explanation for these results other than the high variations of the adhesion of matrix and aggregate. One stone, with building-sand matrix, gives concrete of greater strength because of adhesion; another stone, with Ottawa sand matrix, gives greater strength for that series for the same reason. Adhesion, in turn, depends upon the substance contained in the stone. Minnesota sandstone is composed of clean and granular grains, giving as nearly as possible perfect adhesion with building-sand matrix; but as far as its strength alone is concerned, it pulverizes when pressed between the thumb and forefinger. As in the case of a bridge-pier support that is made to rest upon a bed of sand, providing there can be no leakage of sand, so, too, the loose grains of Minnesota sandstone can be made to withstand a greater load than a hard granite, providing the adhesion is such that there is no leakage of the minutest grain. The moment the bond breaks there is a snap like a broken stick, the specimen failing completely; but when the bond of adhesion does not break until high testing values are reached, as the experiments show, there is no leakage until those points are reached, and the concrete is capable of withstanding a great pressure.

Hence it is believed that the saying, "The stronger the aggregate, the stronger the concrete," is true in only a limited sense; and that these experiments establish the fact that in many cases, stones, which were considered unfit for concrete, can be used to produce a better grade of concrete than a more expensive imported stone will make.

Powder-Hoist Outfit Inspected for Safety Features

The city engineer of a large city in the far West sends a report submitted by one of his assistants who was delegated to inspect the outfit used by a well-known powder company in handling explosives at its property on the waterfront. The following is a condensation of the report, names and location being omitted:

As per your request, I have examined the hoisting apparatus used by the ——— Powder Co. in hoisting powder from barges to the wharf at its docks and fully concur with Captain ———, of the Harbor Patrol, that it is unsafe and may at any time cause the explosion of an entire barge of powder.

During the four years I was United States Deputy Mineral Surveyor for Colorado I had the pleasure of examining many hoisting plants, but cannot recall having seen any quite as crude as the one in question.

The engine is a 2½-hp. gasoline engine, which exhausts directly to the engine room and within 4 ft. of the gasoline-supply tank, which is a large tin can entirely inclosed by the surrounding atmosphere. Should the engine fail to operate at a time when a load of powder is raised 22 ft., there is no adequate means to prevent its falling back upon the powder-laden barge, in which event, should it explode, the laborers would all be subject to being docked for the time spent in midair and actually off the job.

At the outer end of the derrick stands a 12x12-in. post, held a little more than vertical by means of timbers, ropes, pins, spikes and laborers. This post is so ingeniously arranged as to turn on a bottom pivot by the aid of man, thus allowing the load, when raised above the dock floor, to be swung in and lowered unsafely to place. The boom—or perhaps I should say boomerang, for it always returns to the low side without assistance—is a beautiful specimen of second-growth fir and speaks volumes for the age to which such timber can be used. It projects about 12 ft. at an angle of from 80 to 100° and is equipped at its outer end with three pulleys, two defunct and one less so. This boomerang is held by a cable fastened to the top of the center post, which cable is still in a good state of preservation. The ½-in. hoisting line, however, shows signs of antiquity and is supplied with a cheap hook to which it holds mainly by force of habit. From this hook is swung the platform cage, also evidently of domestic design, swung by two ropes fastened to the wooden cage by fence staples and crooked nails. On these platforms, I am told—for I had no desire to see—the men raise eight 50-lb. cans of powder to each load.

This infernal machine is located about 300 ft. from one of the stone piers of the railroad bridge which spans the river at that point, and only a few miles from our own City Hall. I heartily recommend that it be closed to future operations, or that some inspector, whose salary is more commensurate with the risk than your humble servant's, be placed in charge.

Columbia Highway Tunnel

By Henry L. Bowlby*

The construction of the Columbia River Highway along the south banks of the Columbia River in Oregon is offering many opportunities for novel engineering structures. Many concrete bridges and viaducts of unusual design have already been built; a number of them were described in *Engineering News* of Dec. 10, 1914, and Jan. 14, 1915.

In Hood River County, 5 mi. west of Hood River City, a barrier was encountered in the form of a solid rock cliff rising abruptly from the river to a height of nearly 500 ft. Here a bench, or saddle, a few hundred feet wide occurs, and the ledge continues to a height of 2,000 ft. or more. This rocky ledge is known locally as Storm Cliff and also as Mitchell's Point.

An old county road existed for many years over Mitchell's Point. It was narrow and very steep, having in one place a 23% grade 400 ft. long. In 1913-14 when the location surveys of the Columbia Highway were made under the direction of the writer, a highway tunnel was planned through Storm Cliff. Its length was to be 400 ft. The solid basaltic rock cliff seemed to offer an excellent opportunity to reproduce in America the Swiss Axenstrasse, but on a grander scale. Comparative estimates showed that a highway around the face of the cliff, with a 400-ft. tunnel and a 200-ft. concrete-viaduct approach, would cost less to build than a road over the bench, or saddle. The latter route was a mile longer on account of the additional rise and the 5% maximum grade.

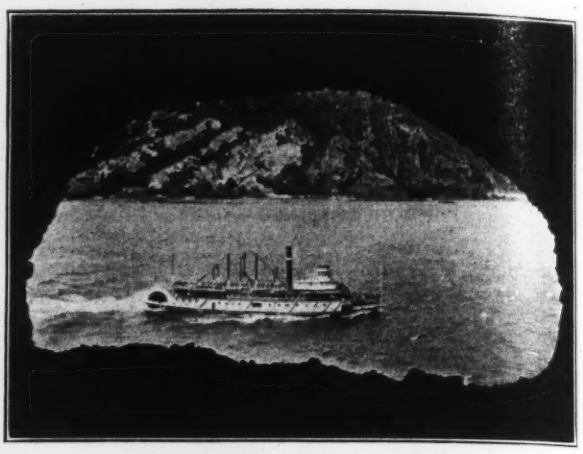


FIG. 2. VIEW FROM WINDOW OF TUNNEL, LOOKING OUT ON COLUMBIA RIVER

In July, 1914, the State Highway Commission promised the citizens of Hood River County that the state would expend $50,000 on the Mitchell's Point section of the Columbia Highway if the county would vote a bond

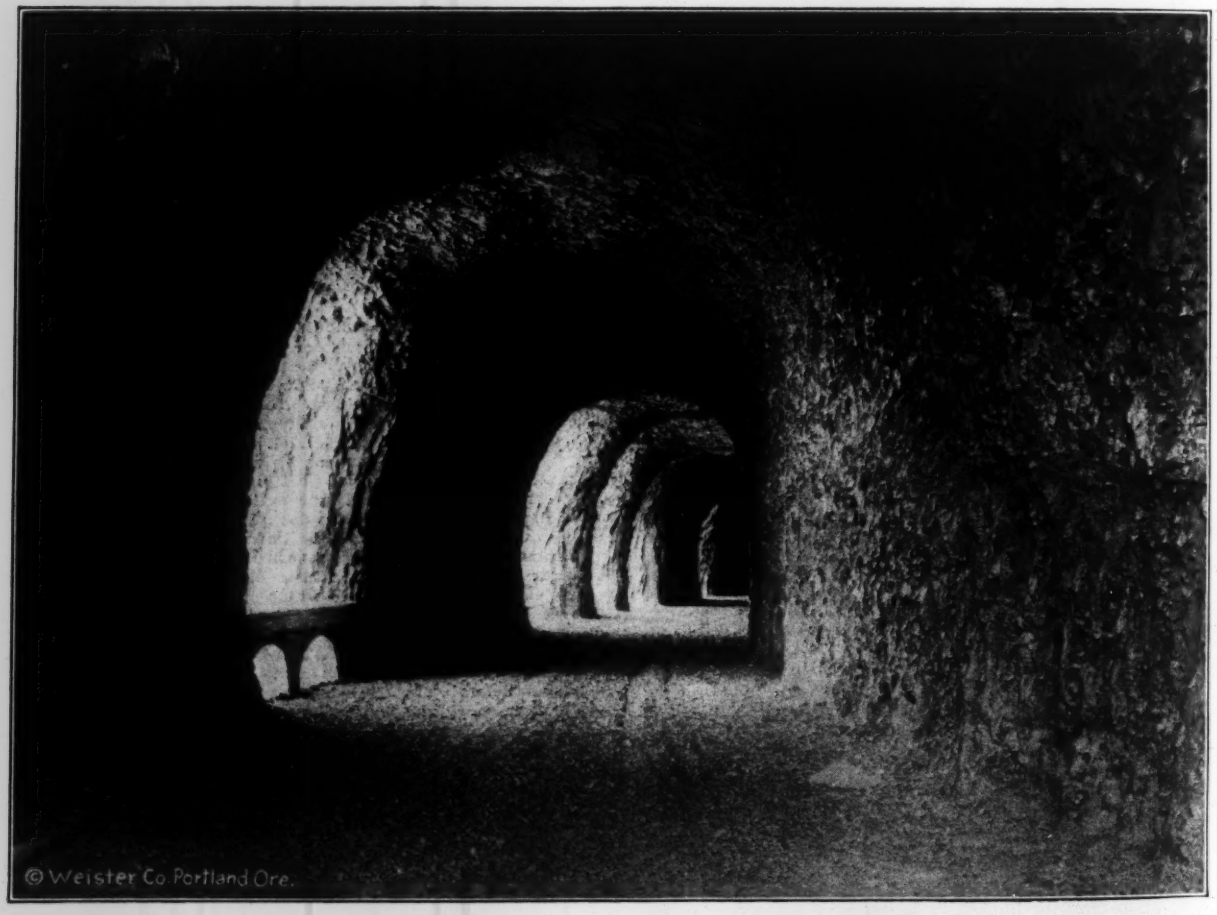


FIG. 1. INTERIOR VIEW OF MITCHELL'S POINT TUNNEL ON THE COLUMBIA HIGHWAY IN OREGON

*Recently State Highway Engineer of Oregon, Portland, Ore.

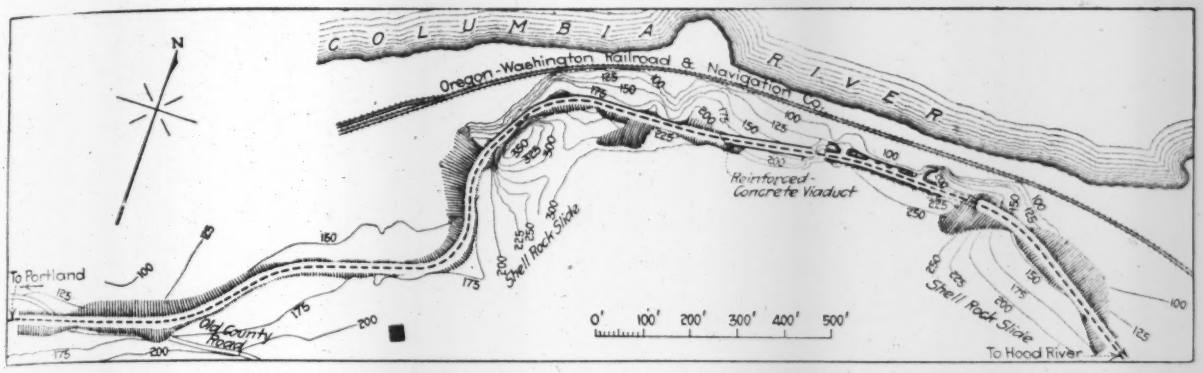


FIG. 3. TOPOGRAPHIC PLAN OF COLUMBIA RIVER HIGHWAY, HOOD RIVER COUNTY, SHOWING STORM CLIFF TUNNEL

issue of $75,000, the proceeds of which were to be used in making the highway passable to the City of Hood River. The bond issue was carried on July 15, 1914, by a vote of 1,502 to 425.

On Mar. 22, 1915, a contract was entered into by the State Highway Commission with the Standifer-Clarkston Co., of Portland, Ore., to build this 3,000-ft. stretch of the Columbia Highway around Mitchell's Point.

The accompanying map shows the topography of the country, and the road as constructed. During construction it was found necessary to make the concrete viaduct 208 ft. long and the tunnel proper only 390 ft. in length.

TABLE 1. FINAL ESTIMATE OF THE CONTRACTOR

4.86 acres clearing @ $90	$437.40
82 sq.rd. grubbing @ $2	164.00
4,284 cu.yd. earth excavation, hauled 0 to 300 ft., @ 24c.	1,028.16
3,734 cu.yd. earth excavation, hauled 30 to 1,000 ft., @ 27½c.	1,026.85
2,394 cu.yd. hardpan excavation, @ 45c.	1,077.30
1,362 cu.yd. loose-rock excavation, @ 47c.	640.14
7,731 cu.yd. shell-rock excavation, @ 30c.	2,319.30
8,898 cu.yd. solid-rock excavation, hauled 0 to 300 ft., @ 77½c.	6,973.45
7,057 cu.yd. solid-rock excavation, hauled 300 to 1,000 ft., @ 81c.	5,716.17
160.25 cu.yd. rubble masonry, @ $10.	1,602.50
339.88 cu.yd. Class "A" concrete*, @ $15.	5,098.20
45.1 cu.yd. Class "C" concrete†, @ $9.	405.90
48,669 lb. reinforcing steel, @ 3½c.	1,703.41
409 lin.ft. concrete railing, @ $1.50.	613.50
74 lin.ft., 12-in. concrete pipe, in place, @ $1.50.	111.00
390 lin.ft. tunnel excavation, @ $33.	12,870.00
30.6 lin.ft. tunnel-window excavation, @ $30.	918.00
119.2 cu.yd. solid-rock overbreak, @ $3.	357.60
Extras, force account.	1,415.08
Total.	$44,477.96

* Class "A" concrete consists of 1 : 2 : 4 mix. † Class "C" concrete consists of 1 : 3 : 6 mix.

TABLE 2. COST OF THE TUNNEL PROPER*

390 lin.ft. tunnel excavation, @ $33.	$12,870.00
30.6 lin.ft. tunnel-window excavation, @ $30.	918.00
119.2 cu.yd. solid-rock overbreak, @ $3.	356.60
20.79 cu.yd. rubble masonry in railways, @ $10.	207.90
5.74 cu.yd. Class "A" concrete, @ $15.99.	86.10
Force account, cementing cracks over portals.	33.23
Total.	$14,472.83
Length of tunnel, ft.	390
Cost per lin.ft.	$37.10

* The cost of the tunnel proper includes windows, concrete railings, and a trail from one window to a point on the face of the cliff, overlooking the outside of the tunnel.

TABLE 3. COST OF THE CONCRETE VIADUCT

300 cu.yd. Class "A" concrete, @ $15.	$4,500.00
45.1 cu.yd. Class "C" concrete, @ $9.	405.90
409 lin.ft. concrete rail, @ $1.50.	613.50
48,400 lb. reinforcing steel, @ 3½c.	1,694.00
355 cu.yd. shell-rock excavation, @ 30c.	106.50
Force account, covering extra excavations for foundations for columns, and placing an expansion joint in the viaduct.	1,230.38
Total.	$8,550.28
Length of viaduct, ft.	208
Cost per lin.ft.	$41.10

The tunnel section is 18 ft. wide with vertical sides 10 ft. high. The heading is semicircular, having a radius of 9 ft. As originally planned, there were to be seven windows, but during construction it was found best to relocate them at certain places, and only five were cut through to the face of the cliff. The windows are approximately 20 ft. wide and 19 ft. high.

The specifications provided for a bonus to the contractor for careful work. It was provided that payment should be made at so much per lineal foot for the neat tunnel cross-section. The contractor was allowed to make the tunnel at any point 5% by area less than the neat cross-section and receive payment for the full cross-section. He was also permitted to make the tunnel at any cross-section larger by 10%, and the payment made at the overbreak was at the same rate of $3 per cu.yd. If, however, the tunnel cross-section at any point was made larger in area than 10% in excess of the neat cross-section, no payment was made for the overbreak, and the contractor received only the amount of his bid per lineal foot for tunnel excavation. In all cases measure-

FIG. 4. STORM CLIFF OR MITCHELL'S POINT TUNNEL, COLUMBIA HIGHWAY

ments were made for the rectangular portion of the tunnel cross-section from the vertical median line and for the tunnel heading from the center of the semicircle.

No lineal dimensions were permitted less than 5% nor greater than 10% over the dimensions required for the neat cross-section. The excavation of the windows was made under similar specifications.

Fortunately the contract was awarded to a very good firm of contractors, who took a great deal of pride in the work, with the result that a job with an excellent finish was obtained. Work was begun Mar. 31, 1915, the road was opened Sept. 6, 1915, and work was completed Nov. 10, 1915.

The contractor's final estimate, the cost of the tunnel proper and the cost of the concrete viaduct are given in the accompanying tables.

The cost of the portals, being work similar to solid-rock excavation, was paid for as such and has not been included in the tunnel cost. The west portal, next to the viaduct, cost $175.15, and the east portal cost $3,969.98.

The view from the tunnel windows, looking down upon the tracks of the Oregon & Washington Railroad and Navigation Co. and out over the Columbia River into the State of Washington, is superb.

This project was planned, surveys made, estimates prepared and the contract let under the direct supervision of the writer, while State Highway Engineer of Oregon. It has been completed under the administration of the present Highway Engineer, E. I. Cantine. J. A. Elliott was locating engineer, had charge of the preparation of the estimates and was district engineer in charge of construction of the project to completion.

Strengthening the Cairo Bridge on Illinois Central R.R.

The strengthening of the Illinois Central R.R. bridge over the Ohio River at Cairo, Ill., is an interesting illustration of the possibility of adapting an old bridge to carry a loading much heavier than that for which it was designed. The Cairo bridge was built in 1887-88, the designer and consulting engineer being the late George S. Morison.

The trusses were proportioned to carry a moving load of 3,000 lb. per lin.ft.; but in calculating the effects of a moving load, the part of any strain in excess of that which would have been produced by a uniform load of equal amount was taken on a basis of 5,000 lb. per ft. The top lateral system was proportioned to resist a wind pressure of 300 lb. per lin.ft., and the bottom lateral system 500 lb. per lin.ft. The floor system was designed for a uniform load of 6,000 lb. per lin.ft. The compressive strain in the top chord was limited to 14,000 lb. per sq.in. of balanced section. The tensile strain in the bottom chord was limited to 13,000 lb. per sq.in., and that in the web members was kept somewhat lower (*Engineering News*, Feb. 7, 1891, and Jan. 11, 1906).

The bridge consists of 34 spans—2 through spans of 518 ft. with double-intersection trusses, 7 similar spans of 400 ft., 3 deck spans of 249 ft., 21 of 150 ft. and 1 of 106 ft. All are of the Pratt type. The weight of the spans is 1,032 tons for each 518-ft. span, 567 tons for the 400-ft., 235 tons for the 249-ft. and 100 tons for the 150-ft. spans. The total length at present is 3,882 ft. for the bridge proper, and 20,461 ft. inclusive of the approaches (steel spans and earth embankments). The rail level is 110 ft. above low (or 55 ft. above high) water.

The substructure consists of stone and concrete piers, the foundations for which were put down by the pneu-

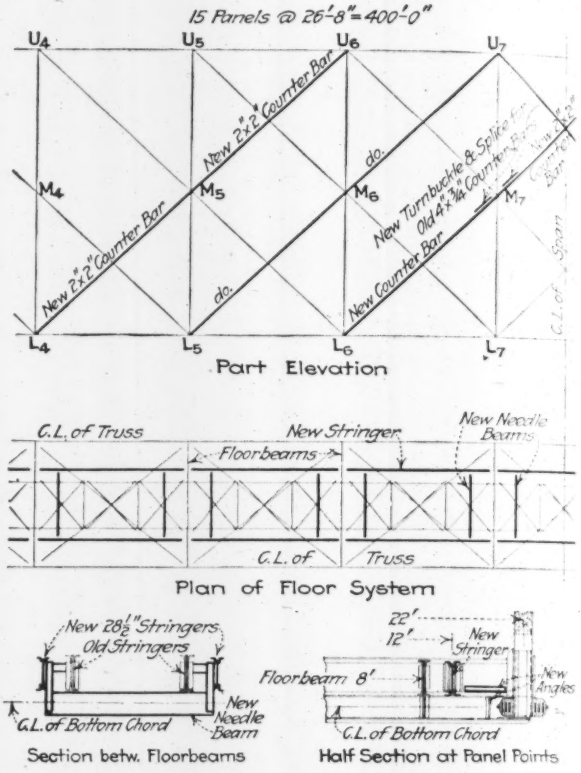


FIG. 1. REINFORCEMENT OF TRUSSES AND FLOOR SYSTEM IN THE 400-FT. SPANS OF THE CAIRO BRIDGE

matic process. The caissons were sunk to a depth of 75 ft. below low water, in a formation of sand and gravel. No alterations were necessary in connection with the strengthening of the superstructure.

Alterations to the Cairo Bridge

The bridge originally had long approaches of steel deck-truss spans on both sides of the river. Owing to the heavy traffic and the double-tracking of the line on each side, it was proposed in 1903 to double-track the approaches and also to build a new double-track superstructure across the river. The reconstruction could have been carried out by extending the piers or by capping them with bearing girders of sufficient length to carry the new bridge seats. This work was not done, except that the steel spans of the Illinois approach were replaced by a double-track embankment, leaving only a 52-ft. girder span and two 249-ft. truss spans between the fill and the 518-ft. channel spans.

Apart from the increase in traffic capacity it was necessary to provide greater strength in the bridge, to accommodate the weight of modern locomotives and trains. This work was carried out during 1914 and 1915. It was done with company forces, using the regular equipment of the bridge gangs. Owing to the great length of the structure, the work was spread out to a considerable extent, and four air-compressor plants were in use

for the riveting. All work was done between trains, and there was very little interruption to traffic.

The bridge as now strengthened provides for a live-load of one 230-ton 2-8-2 locomotive followed by 80-ton loaded cars (55 tons' load per car). The design and execution of the reinforcement were under the direction of A. S. Baldwin, Chief Engineer, Illinois Central R.R., and F. L. Thompson, Assistant Chief Engineer.

Strengthening the Truss Spans

The trusses of the through spans were strengthened by the addition of counters consisting of square bars 2x2 in. Each bar was attached to the pin at the the lower chord, and at the center of the post by means of U-shaped bars with clevises. At the top chord the connection was a stiff bracket riveted to the under side of the chord and the face of the post, the counter being attached to this bracket by means of a clevis and pin. This arrangement is shown in Figs. 1 and 2.

The floor system of the 250-ft. deck spans and of the longer through spans was reinforced by placing additional stringers outside of the original track stringers. At the quarter points of each panel, transverse needle beams beneath the floor were suspended from the new stringers. These beams had shoes beneath the old stringers, and by means of wedges the latter stringers were brought to a bearing on the shoes. In this way the effective length, or span, of the old stringers was reduced.

In the 518-ft. spans the new stringers are 30½-in. plate girders 30 ft. 3¾ in. long. The web plate is 30x ⅜-in., and the flange angles are 6x3½x¾ in. On account of the hangers for the needle beams the bottom inner flange angle is omitted and a bar 6x½ in. is used in its place (Fig. 2). The needle beams for these spans are 20-in. 60-lb. I-beams.

It was desired to have the new stringers of uniform length, so far as possible, in order to permit of their easier erection, and also to meet the variation in distance

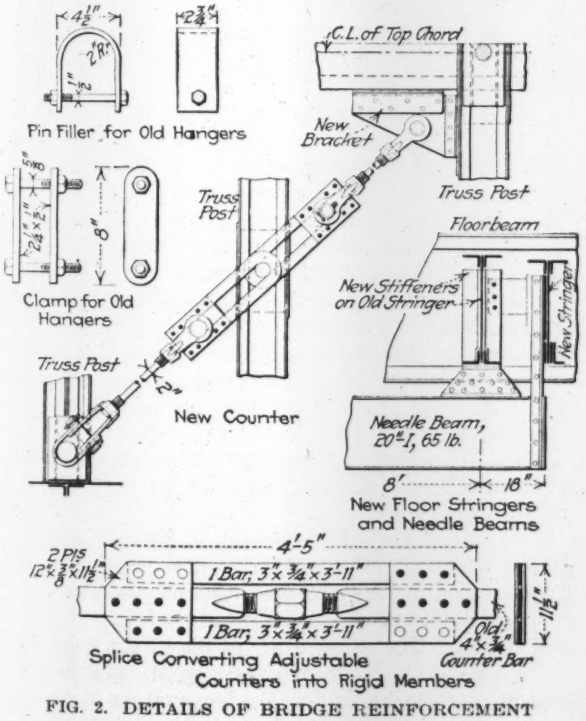


FIG. 2. DETAILS OF BRIDGE REINFORCEMENT

between the faces of the floor beams, filler plates of different thicknesses were supplied, to be placed between the ends of the stringers and the webs of the floor beams.

The 150-ft. and 106-ft. deck spans were reinforced by adding a pair of 24-in. I-beams beneath each track stringer, the latter resting on 7x16-in. wooden blocks 28 in. long, which were laid across the I-beams at the third points of each panel. The light top lateral rods, which had been kept in adjustment with considerable difficulty were replaced by rods of heavier section. Sway bracing was added in the vertical plane of the truss posts. The deck stringers and bracing are shown in Fig. 3.

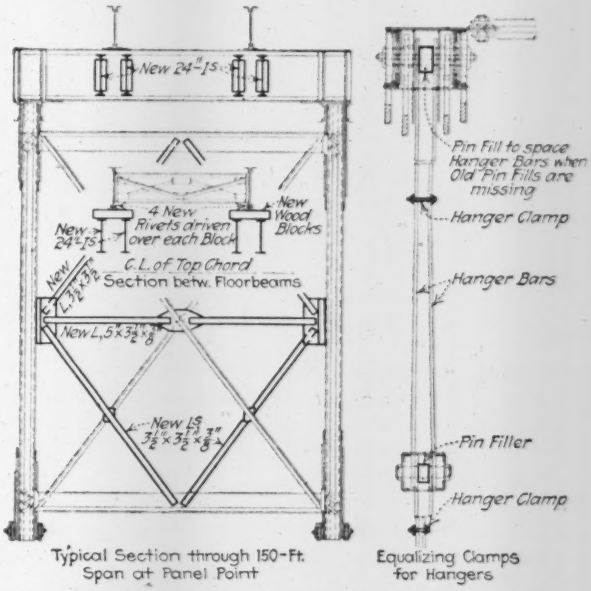

New 24"I's
4 New Rivets driven over each Block
New Wood Blocks
C.L. of Top Chord
Section betw. Floorbeams
New L, 5"x3½"x⅞"
New L's 3½"x3½"x⅞"
Typical Section through 150-Ft. Span at Panel Point
Pin Fill to space Hanger Bars when Old Pin Fills are missing
Hanger Clamp
Hanger Bars
Pin Filler
Hanger Clamp
Equalizing Clamps for Hangers

FIG. 3. REINFORCEMENT OF 150-FT. SPANS AND METHOD OF EQUALIZING STRESSES IN HANGER BARS

All adjustable members of all the spans had been provided with closed sleeve nuts. As the threads in many of these nuts were in bad condition, it appeared that it would be impossible to maintain proper adjustment and that a dangerous condition might result. A detail was devised therefore whereby the sleeve nuts were adjusted and a rigid splice connection then placed over them, thus converting the member from an adjustable to a rigid member. This construction is shown in Fig. 2.

In many of the vertical hangers consisting of two bars it was found that the adjustment in the bars was not uniform. To correct this defect, a clamp was applied consisting of two flat bars connected by bolts (Figs. 2 and 3). This method proved to be a very effective way of equalizing the adjustment in the two bars.

Field measurements were taken in all cases, and no trouble was experienced in fitting the parts in place.

⌘

The Chicago Railway-Terminal Problem is again to be taken up by the city with a view to the concentration of the present passenger-terminal stations. Under ordinances passed some time ago the Illinois Central R.R. is required to build a new terminal station on a site adjacent to the present station, in order to make way for the extension of 12th St. to the lake front and for the improvement of Grant Park. The council committee on railway terminals has now been directed to confer with the several railways entering from the south in order to determine upon the exact location of the new Illinois Central R.R. terminal and to provide for its use by other roads which now run beyond 12th St. and use the Dearborn, La Salle and Grand Central terminal stations.

Tile Drainage by Day Labor and by the Rod

SYNOPSIS—Drainage system for farm lands, on Mississippi River, being built under supervision of engineers. How the concrete drain tile is made; costs of pipe. Work done both by day labor and by piecework. Prices paid per rod.

The system of tile drainage for an area of about 1½ sq.mi., which is shown in Fig. 1, is designed for the complete or intensive farm drainage of land already provided with ditches for general drainage. Landowners and farmers are coming to realize the value of this kind of work in maintaining the land in good condition for agricultural use. What is still more important, they are coming to realize the necessity of carrying out the work in a comprehensive manner instead of each man building a small tile system for himself, to discharge the water onto adjacent land that may or may not be drained.

The section referred to is a part of the Bay Island Drainage and Levee District, in Mercer County, Illinois. The district has an area of about 33 sq.mi. and is protected from the Mississippi River by a levee along the west side. Its dredged ditches have a flow to the south and are intended to drain the lakes and sloughs. At the outlet of the main ditch the water is pumped over the levee into the river.

Some tracts of land within the district have scattered lines of tiling for individual drainage, but these are not laid out in a comprehensive system and are quite inadequate for thorough drainage. The aim in these cases has been merely to drain low spots; so while the open ditches are of some use in a dry season, they are practically useless in dealing with water coming from adjoining lands or with the overflows from the lakes.

The section shown on the map includes tracts of land owned independently by the Mueller Lumber Co., Dr. Bomke and Ziegler Brothers, but its drainage is planned as one comprehensive system. The boundaries of the drainage area were determined and the main drains then located through the low points. The laterals are spaced uniformly 100 ft. apart. In the area of 1½ sq.mi. there will be about 75 mi. of 6- to 22-in. tile, with two 22-in. and eight 12-in. outlets into the open ditch.

The runoff is taken as ½ in. in 24 hr., this rather high figure being used as there is some seepage from the river during stages of high water. The soil is rich loam, with clay subsoil and some streaks of sand.

Trenches for Tile Drains

The trenches will be 3½ to 7 ft. deep. The excavation is done by hand, each trench being commenced at its lower end. The lines are staked out and the trenches excavated about 10 in. from the stakes. The top soil and clay are thrown out on opposite sides. Care is taken not to cut below the grade line, and the bottom of the trench is grooved and shaped to receive the tile and hold it in position.

In connection with the excavation it was required that all stumps, trees and brush within 30 ft. of the drains should be removed. The specifications called for grubbing out the roots to a depth of at least 3 ft. Different plans were adopted, however, by which a portable saw is used for cutting the trees, and stumps are blown out with dynamite where necessary, as shown in Fig. 2. The wood is either burned, or stacked for future use.

To give the grade, transverse frames are erected at the grade stakes, over which a cord is stretched above the center line of the ditch and at the required grade. This is shown in Fig. 2. These supports must be kept erected for at least three grade stakes and the work checked by sighting over them. Intermediate supports are placed to support the cord at about 50-ft. intervals, carefully lined in by sighting.

Concrete pipe is used. It is in lengths of 12 in. for pipe 12 in. in diameter and under, 18 in. for 15- and

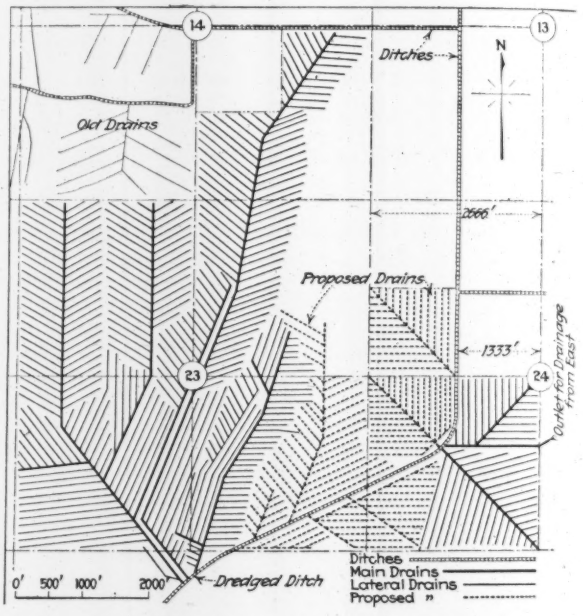


FIG. 1. MAP OF A TILE-DRAINAGE SYSTEM IN ILLINOIS

18-in. pipe, and 30 in. for 22-in. pipe. The laying is commenced at the lower end of each drain. In putting in the pipes they are placed until the top of the joint is closed, except that in sand or fine silt the bottom of the joint must be laid close to prevent the material from entering. In the latter case the top of the joint is covered with clay. Where an opening of ¼ in. or more is left in order to make turns or on account of irregular tile, this gap must be covered with broken pieces of pipe. Fig. 2 shows ready for shipment some of the pipe, which is supplied by the Ziedler Concrete Pipe Co., of Muscatine, Iowa.

The pipe is made by a Schenk & Stewart revolving-process machine, using washed and screened sand and gravel graded in four sizes and obtained from the Northern Granite Gravel Co., of Muscatine, Iowa. A 4-min. mix is used. The temperature of the mix is maintained at from 80° to 90° F. After casting, the pipe is placed in a vapor room for 36 hr., the temperature being 110°.

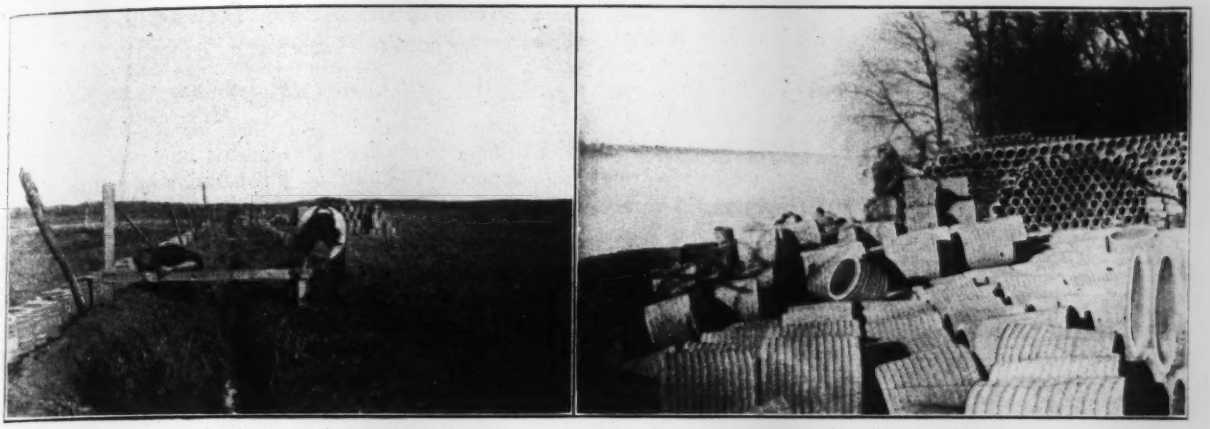


FIG. 2. TILE DRAINAGE IN THE BAY ISLAND LEVEE AND DRAINAGE DISTRICT IN ILLINOIS
Left—Setting the cord for line and grade. Right—Concrete tile

The cost of the pipe per thousand feet, f.o.b. Muscatine, is $17.50 for 4-in., $22 for 6-in., $42 for 7-in., $52 for 8-in., $72 for 10-in. and $94 for 12-in.

After the drain has been inspected and its grade checked, the pipe is covered, or blinded, with about 6 in. of clay before backfilling. When the pipe is secure and the soil favorable, this blinding may be omitted; but in no case is sand allowed to be filled directly upon the tile. The backfilling is done by hand.

Engineering and Organization

The work is being done mostly by piecework, under the direction of the Central States Engineering Co., of Muscatine, Iowa, of which C. H. Young is Chief Engineer. It constitutes a joint job for the three owners already mentioned, who deal directly with the chief engineer. It was intended originally to do the work by the rod or day, but the plan was abandoned in favor of piecework. The tile is hauled both by day labor and by the ton, the company building necessary roads and bridges from the river bank to the site. Clearing is done by the day and by the acre, the company furnishing dynamite and a small portable sawmill. The temporary pumps are operated by the company.

The engineer in charge in the field has full authority to lay out and direct the work and supervise the construction. He has charge over the timekeeper, the general foreman and his crew, the cook and his assistants. He directs the placing of material, boarding and lodging the men, setting grades and line, etc. He countersigns all pay-checks and approves all bills. A checking account is placed at his disposal for unforeseen emergencies and for paying off employees in case of discharge. He determines the number of men to be employed to carry on the work properly and has authority to discharge any employee.

The timekeeper keeps the time, makes out payrolls, takes care of the commissary and generally assists the engineer. The general foreman has charge of the laborers (both on the work and in the bunkhouse). He signs the teamsters' haulage tickets and keeps a check list of his men. He has authority to employ or discharge laborers, but the number of men to be engaged at any time is determined by the engineer in charge.

Men hauling material (whether by the day, load or ton) must secure from the timekeeper a ticket showing the number of pieces of each size of pipe hauled along the ditch, payment being made only on the basis of these tickets, which are described later. At present nearly all the tilers are employed by the rod, as the men will work harder and make more money if there is some inducement. The overhead expenses and distribution of material are handled by the day.

The pay of this working organization is as follows: Engineer in charge, $7 per day and expenses; timekeeper, $50 per month, with board and lodging; general foreman, $90; cook, $60; cook's helpers, $35. For hauling material a uniform rate of $1 per ton has been agreed upon by the employers, who undertake to deliver material along the trenches on their respective properties.

Costs and Accounts

The question of ditching by machine was considered, but the cost by this method promised to be high. The soft ground made it necessary either to wait until the swamp was frozen or to mount the ditcher on skids. Bids were called for, but the contractors did not seem enthusiastic over the job; they wanted to lease their ma-

Daily Report

CAMP ZEIGLER | STATE | DATE 191

Occupation	Name	Excavation						Amt.	Location					Hrs.	Pipe			Misc.	Meals	Supplies		Remarks
		Hrs. Toping	Hrs. Bottom	Rate Per Rod	Rate Per Day	Rate Per Mo.			System	Sub. Main	Lateral	Station From	Station To	Backfill	Hauling Pcs.	Hauling Size	Hauling Ton		No.	Amt.	Item	

FIG. 3. FORM OF DAILY REPORT FOR TILE-DRAINAGE WORK

chines at a definite rate. Following are the prices in cents paid per rod and the width of the trench in inches:

	6-In.	Width of Ditch	8-In.	Width of Ditch	10-In.	Width of Ditch	12-In.	Width of Ditch	15-In.	Width of Ditch	18-In.	Width of Ditch	22-In.	Width of Ditch
Topping....	12	5 14	12	5 14	15	0 20	15	0 20	17	0 22	20	26	35	31 5
Bottom....	22	5 14	22	5 14	32	5 20	37	5 20	45	0 22	55	26	67	31 5

Overdepth paid for at 1.5c. per in. of depth.

These prices are computed on the maximum basis of 2.5c. per in. in width of ditch for bottom work and 1c. per in. of top width of ditch. An 18-in. spade depth is given as the depth for topping and bottom work. As the digging was easy, the prices were reduced slightly, so that a 14-in. ditch, for example, on topping cost 12.5c. instead of 14c. The price will not exceed 2.5c. for bottom and 1c. per top.

CAMP ZEIGLER Date..... 191....

Mr..... Teamster.....

	Pcs.	Size	Tons	System	Lateral	From	To	Remarks
1st Load								
2nd Load								
3rd Load								
4th Load								

TOTAL_____ Amt. $_____

By _____

FIG. 4. FORM OF HAULAGE TICKET

The pipe is floated about 10 mi. down the river on barges and distributed by wagons, the average haul being 2½ mi. Much of it will be distributed during the winter while the ground is frozen. For the working forces a camp has been built, with bunkhouses, messroom, etc. About 40 men will be employed when work is in full swing.

The expense accounts are divided under five general classifications—materials, construction, board and lodging, small contracts, engineering. The blank forms used in keeping the records were designed specially for this work. The daily report, Fig. 3, is 10¾ in. wide and 7½ in. high. The haulage ticket, Fig. 4, is 3x6 in., and the payment order, Fig. 5, is 5½x5½ in.

CAMP ZEIGLER Date.....

Mr.....

Please give Mr..... .....

the following amount for:

Topping.....rds. at.....$.....between station.....and station.....
.....on.....system.....lateral.

Topping.....rds. at.....$.....between station.....and station.....
.....on.....system.....lateral.

Topping.....rds. at.....$.....between station.....and station.....
on.....system.....lateral.

Bottom.....rds. at.....$.....between station.....and station.....
on.....system.....lateral.

Bottom.....rds. at.....$.....between station.....and station.....
on.....system.....lateral.

Bottom.....rds. at.....$.....between station.....and station.....
on.....system.....lateral.

Clearing trees for Mr..... .....at.....$.....

Hauling.....tons of tile at.....$.....

Labor for.....at.....hours.....$.....

Labor for.....at.....hours.....$.....

Deduction for board.....meals at.....$.....

" " Comissary.....$.....

.....
Timekeeper or Foreman

Received payment in full to date.

.....
(Signed)

FIG. 5. FORM OF PAYMENT ORDER

Water Pressure Raised and Waste Survey Made*

By Francis F. Longley†

St. John's, Newfoundland, has for some years past been troubled with deficient quantity and pressure in its water-supply. The city has a population of a little more than 30,000. It has taken its water-supply for many years past from four or five miles west of the city and about 500 ft. above sea level. From the easterly end of the lake a conduit extends about two miles to a basin. From this basin two pipe lines, mainly 16 in. in diameter, extend to the city by two different routes.

The most obvious defect in the entire system at the time we [Hazen, Whipple & Fuller] were invited to take up this study was the inadequacy of the distributing mains in the city. The water consumption averages about 6,000,000 U. S. gal. a day. The system is extremely weak in large mains, resulting in a great loss of pressure, especially at times of maximum consumption.

Winter conditions have always imposed a severe drain on the water-supply, as the construction of the houses and the design of the plumbing and the methods of heating in common use do not tend to prevent the freezing of the pipes within the houses, and the water is quite commonly permitted to run, to prevent freezing. The cold-weather draft of water, therefore, runs up very high, and at these times the pressure in the city has at certain times and certain places almost disappeared. Besides the reduction of waste, the thing that was obviously needed was a few main arteries that would carry the quantity of water demanded by the city to the most remote parts with a minimum loss of head. The work of laying these new mains went forward during the summer of 1915, and at the end of the working season most of the mains were ready to put into service.

Leakage and Pressure Tests

A series of tests were at that time made upon the improved system. The first was a leakage test. Some 8,000 ft. of 20- and 16-in. mains was tested under approximately the normal working pressure, and found to give a leakage not exceeding about 2 gal per min., indicating that the work had been well done. Then fire-stream tests were made, to show the improvement secured by the addition of these mains in the pressures over the lower part of the city.

As the system stood before the new mains were connected with the old ones, the pressure along Water St. was drawn down by 11 fire streams to about 36 lb. After the new mains had been connected to the old ones, 11 fire streams reduced the pressure to only 68 lb. The pressure that yielded four fire streams in the old system now yields ten streams; the pressure that yielded three fire streams in the old system now yields eight; the pressure that yielded two fire streams in the old system now yields six. Further benefits will follow the laying of the additional supply main from the source and the connection with the new service reservoir.

The intake of the system at Windsor Lake is quite inadequate. One of the most serious intake difficulties

*Condensed from a paper read before the New York Section of the American Water-Works Association, New York City, February, 1916.

†Of Hazen, Whipple & Fuller, Consulting Engineers, 30 East 42rd St., New York City.

with ice. In order to eliminate these difficulties and to provide means for maintaining an adequate supply under all conditions, plans have been made for a new intake of 36-in. steel pipe extending some 600 ft. more or less into the lake.

In the past the city has had no means of measuring the quantity of water used. Some measurements were made for the first time in the fall of 1914, by means of a weir. Plans have been made for a venturi meter, gate house and connections, which will afford complete means for the measurement and control of the flow.

St. John's has had no service reservoir in the past, depending upon the flow through the long supply lines to take care of the maximum as well as the ordinary draft. A natural lake, known as George's Pond, lies in an admirable location and at a most suitable elevation for a service reservoir. George's Pond has an area of about 13 acres and a good depth. It lies at an elevation slightly higher than 300 ft. above sea level and at a distance of only a few thousand feet from the eastern end of the city. The elevation of the pond is such as to give good pressure for domestic and fire service along the waterfront and for a considerable distance back into the city, and it gives hydraulic conditions that fit in well with the elevation of the main supply.

A 20-in. pipe is being laid to this pond from the mains in the city, thus affording an economical arrangement for a large reserve supply close to the city and securing also a considerable saving in the capacity required in the main supply lines to take care of the maximum demand.

Night Tests Show Great Waste of Water

The weir measurements that were made when this study was first taken up indicated that during that period of a week the average rate at which the water flowed to the city was 6,360,000 U. S. gal. per day, representing a per capita consumption of about 200 U. S. gal. The minimum rate during the 24 hr. was over 80% of the maximum. There are but few manufacturing or commercial establishments that consume any considerable quantities of water regularly at night, and it is therefore probable that the waste is constantly going on day and night at a rate somewhat approaching the observed night flow.

Much of this large amount of waste could be detected and eliminated, if the problem were properly attacked. The most effective way of getting at this would be by meters. Their use was recommended.

From the magnitude of the waste it appeared that something might be done rather rapidly to indicate where the loss mainly occurred, by means of a subsurface study of the distribution system. We therefore undertook to isolate various parts of the distribution system and to get a rough measurement of the night flow into such districts, assuming this night flow to represent approximately the leakage therein. Suitable equipment for convenient and accurate measurement was not at hand, and it was therefore necessary to improvise a method with the material available. The first plan was simply to connect a hydrant within the isolated district to one outside, through a fire hose of known length and diameter, and to measure the drop in pressure between the two hydrants, making proper corrections for differences in elevation. The quantity flowing was then roughly estimated from the friction loss through this hose. This procedure was obviously a preliminary step only, but it resulted in showing in what districts to look for large waste, and it gave a rough approximation of the quantity of leakage therein.

Later, a meter was inserted in the hose line. Careful observation of the meter gave indications of changes in the rate of flow, and this method was used to follow up the first method. The isolated district was then reduced in size by successively closing all available gates, while the meter was under continual observation, and this resulted in a much closer localization of the waste.

From these rather rough methods it was made clear that from 75 to 80% of the total night flow in the districts examined was on the house services and only 20 to 25% on the mains. This determination, of course, was followed up as vigorously as possible by a house-to-house inspection and produced results that have encouraged the city to plan further and more thorough work along similar lines.

The apparatus employed for use in waste reduction consists of two small venturi-meter tubes—one 3x1½ in., the other 2x⅝ in.—erected on the same stand and connected by suitable pressure pipes and valves to a monometer with two scales, one for each of the meter tubes. By means of this apparatus inserted in the hose line connecting two hydrants, one within and one outside the isolated district, very small changes in rate of flow into the district may be observed, yielding information that will permit the elimination of even small amounts of waste on the various services.

Engineering Ethics

That engineers are wage earners and must organize if they are to improve their condition was a view taken in the report of a special committee on professional ethics and standard fees, presented at the annual meeting of the Iowa Engineering Society at Des Moines, Iowa, Feb. 17, by K. C. Gaynor, of Sioux City, Iowa.

It was stated that the codes of ethics of the American Society of Civil Engineers and the American Institute of Architects are well and good for men in consulting work, but that they hardly apply to the much larger number of men in ordinary practice or working for a salary. Probably 90% of the members of the engineering profession work for a wage or salary, while 90% of those of the legal and medical profession work for a fee. Engineers on extensive and costly drainage projects in Iowa work for $5 to $7.50 per day, which is about what a good bricklayer gets. It was suggested that the organization of the American Association of Engineers indicates a recognition of the fact that engineers as a whole form a laboring class.

Another difficulty is that many engineers run contrary to the fundamental principles of ethics or right action. Thus a city engineer, county engineer or college professor, having a sure salary sometimes takes outside work at a price lower than the standard in the community and lower than other engineers can afford. Many material and supply firms also offer free advice and engineering assistance. All this action lowers the price of engineering work and lowers the standard of engineering and can be fought only by publicity.

According to the report, wage-earning engineers must organize if they wish to accomplish any improvement in their condition. If they are too proud to organize, they will have to get along as best they can.

Some Recent Legal Decisions of Interest to Engineers

Municipal Authority to Erect Poles and Wires—The New York Public Service Commission has been recently overruled by the New York Supreme Court in its refusal to grant to the Niagara & Erie Power Co. permission to construct and maintain a power plant in the town of Portland. The power company had made application to the Portland town board to construct poles and wires within the town. The permission was granted. The company then petitioned the Public Service Commission for permission to construct its plant in the town. This was denied upon the sole ground that the company had not obtained consent of the town superintendent of highways. The commission took the position that the New York transportation law requiring the consent of the "municipal authorities" placed a duty upon the corporation to secure the consent of the highway superintendent. The Supreme Court in overruling the Public Service Commission (Farnsworth vs. Boro Oil and Gas Co.; 156 N. Y., 879), said:

> The question before us is, Who are the "municipal authorities" of a town within the meaning of this expression? The truth is, very likely, that the legislature did not have any particular official or body in mind. It is simply one of those careless, indefinite expressions which so frequently creep into statutes. But it is not so important in this case to discover the legislative intent as to avoid a conflict of authority when applications like the one before us are made. It is desirable that the responsibility of granting or refusing applications for franchises of this nature in towns be definitely fixed and placed upon some specific body or official.
>
> This question has never been directly passed upon by the courts, so far as we can discover, although there have been several expressions pro and con. The latest, highest and most authoritative expression shows clearly that the Court of Appeals considered that the town board is the "municipal authorities" intended by the statute.

Purchaser Using Machine That Fails to Meet Test Must Pay for It—A contract for the sale of an engine contained certain guaranties and provided for an endurance test, by means of which the engine should be shown to possess the qualities specified, and that, if the engine did not fulfill the test or satisfy the guaranties, the purchaser should not be required to pay therefor, and the engine should be forthwith removed. The test left the question whether the engine fulfilled the guaranties in dispute, and the purchaser, though continuing to use the engine, refused to accept it or pay for it.

An action was brought for the price of the engine by the manufacturer, and it was held that a recovery for the purchase price should be allowed. Although the engine failed to meet the test to which it was subjected, the court held that the continual refusal of the purchaser either to accept or return the engine made him liable for the purchase price. Keeping and using the engine caused title to pass to him, and from that time the guaranties ceased to be conditions precedent and became collateral agreements, for the breach of which the purchaser was confined to a recoupment of damages, as his conduct in continuing to use the engine was conclusive evidence that the engine was a substantial performance of the contract. (Crescent Milling Co. vs. Straight Manufacturing Co.; 225 Fed., 808).

Operation of Telegraph Lines Over Post Roads—A telegraph company that has accepted the provisions of the acts of Congress giving the right to construct, maintain and operate telegraph lines over and along any of the military or post roads of the United States is protected by that act against exclusion or other arbitrary interference by a town whose authorities, with full knowledge of all the circumstances, permitted the location and construction of such lines along the highways and for more than 20 years acquiesced in their maintenance and operation, although the telegraph company was never granted the formal antecendent permit contemplated by state law "specifying where the posts may be located, the kind of posts and the height at which and the places where the wires may run." (New England Telegraph Co. vs. Massachusetts; 36 Sup., 103.)

Creosoting Wood Blocks in Vertical Tanks

By E. A. Sterling*

A comparatively new idea in the process of creosoting wood block has been developed, which is of great importance to the lumber industry, because it promises to reduce the cost of wood blocks materially. This idea is mainly one of economy in operation and does not involve any new or untried methods of impregnation or preservative material. Two engineers, working independently, evolved practically the same fundamental idea in the matter of vertical treating cylinders. The first to design and build a plant of this character was J. B. Card, Chicago, followed by G. B. Shipley, Pittsburgh, who is more recently in the field with a new plant embodying all the improvements and advantages that present experience makes it possible to apply to the vertical-cylinder idea. This plant is described herewith.

The vertical wood-block cylinders at Orrville were installed by the Ohio Wood Preserving Co. in conjunction with its horizontal cylinder, which is of the usual type for treating ties and other classes of material. Each type of cylinder is equipped for the use of any standard process using either creosote or zinc chloride; but because of the greater economy through the elimination of the wood-block cages necessary in the horizontal cylinder and the mechanical handling of the blocks from the planing mill to the cars, the vertical cylinders have been devoted so far entirely to wood-block treatment. The boiler house, horizontal-cylinder building, vertical-cylinder equipment, planing mill and shop are all housed in separate buildings. Steam is generated in two boilers of 150 hp. each, with steam at 100 to 150 lb. working pressure, and so fitted that coal, wood, refuse or gas can be used as fuel at any time. Electric current is furnished by a local power plant, and all the wood-working machinery is driven by direct-connected alternating-current motors, while the crane is driven by direct current.

The vertical cylinders for treating paving blocks are of two sizes, one holding 300 yd. and the other 200 yd. Each cylinder is fitted on the inside with a hydraulic plunger and platform so that, when empty and about to be charged with wood blocks, the platform is at the top. As the blocks are delivered into the tank, the platform lowers gradually, thus preventing breakage. When the cylinder is full, the top cover, or door, is put on; and the blocks are treated in the usual manner. After treat-

*Manager, National Lumber Manufacturers' Association, Chicago.

ment when the cover is taken off and the plunger started, the blocks are forced up and discharged into a chute that leads directly into cars for shipment.

The advantages of treating wood blocks in a vertical type of cylinder may be summarized as follows:

1. Reduction in labor, owing to the mechanical handling of the blocks from the saws to the outbound cars.
2. Lost space is reduced to a minimum, permitting in each charge more cubic yards of blocks per unit of cylinder displacement than is possible in a horizontal cylinder, where block baskets or cages are necessary.
3. The vertical cylinders require only one-tenth of the ground space occupied by the horizontal plant.
4. The first cost of the entire equipment is materially less than that of a treating plant with horizontal cylinders.

Perhaps the chief advantage of the vertical-block plant from the standpoint of the lumber industry is its simplicity of operation and its low first cost. Only two machines are required and much smaller tanks and cylinders, thus permitting the construction of paving-block plants in connection with sawmills or near consuming points. Freight rates on shipments of blocks to remote territory may thus be eliminated, rendering it easier to meet the competition of other paving materials. In many localities the cost of wood blocks has been almost prohibitive on account of the long haul—a factor which can be largely eliminated if inexpensive vertical treating plants can be erected in regions that it is now possible to serve only from distant plants. Another consideration is that the capacity of the plant may be more readily fitted to the needs of the locality, the size and number of cylinders being readily adjusted or changed.

The vertical type of plant is also adapted to treat material other than blocks, so that local requirements may be taken care of or material treated for retail trade, such as silo stock, bridge material and farm timbers. The cylinders are operated by combined air compressor and vacuum pump, thus reducing all piping to a minimum. All measuring tanks are also vertical and mounted on weighing scales, so that the exact amount of preservative absorbed by the wood is determined directly instead of by the old volumetric measurements, which required temperature adjustment.

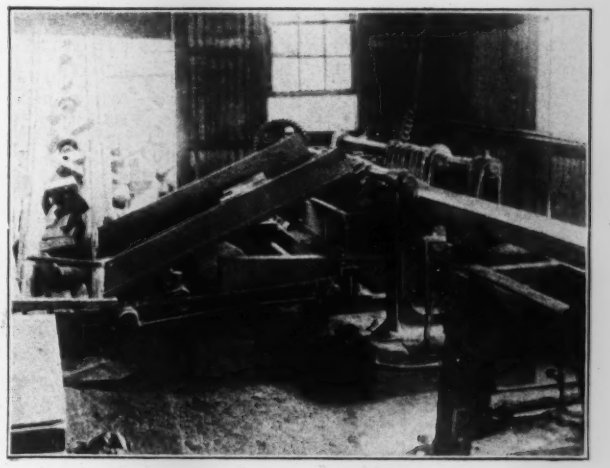


FIG. 2. WOOD-BLOCK SAW AND LOWER END OF AUTOMATIC CONVEYOR

The blocks are cut by an automatic band-saw (Fig. 2) running on ball bearings and driven by a direct-connected motor. The machine can be adjusted for different capacities, although it is generally used with eight saws, which easily cut 70 yd. of blocks an hour.

Another interesting feature of the Orrville plant, which has nothing directly to do with the wood-block treatment, is the use of a fireless locomotive for switching cars and tie trams in the yard. This locomotive operates by steam,

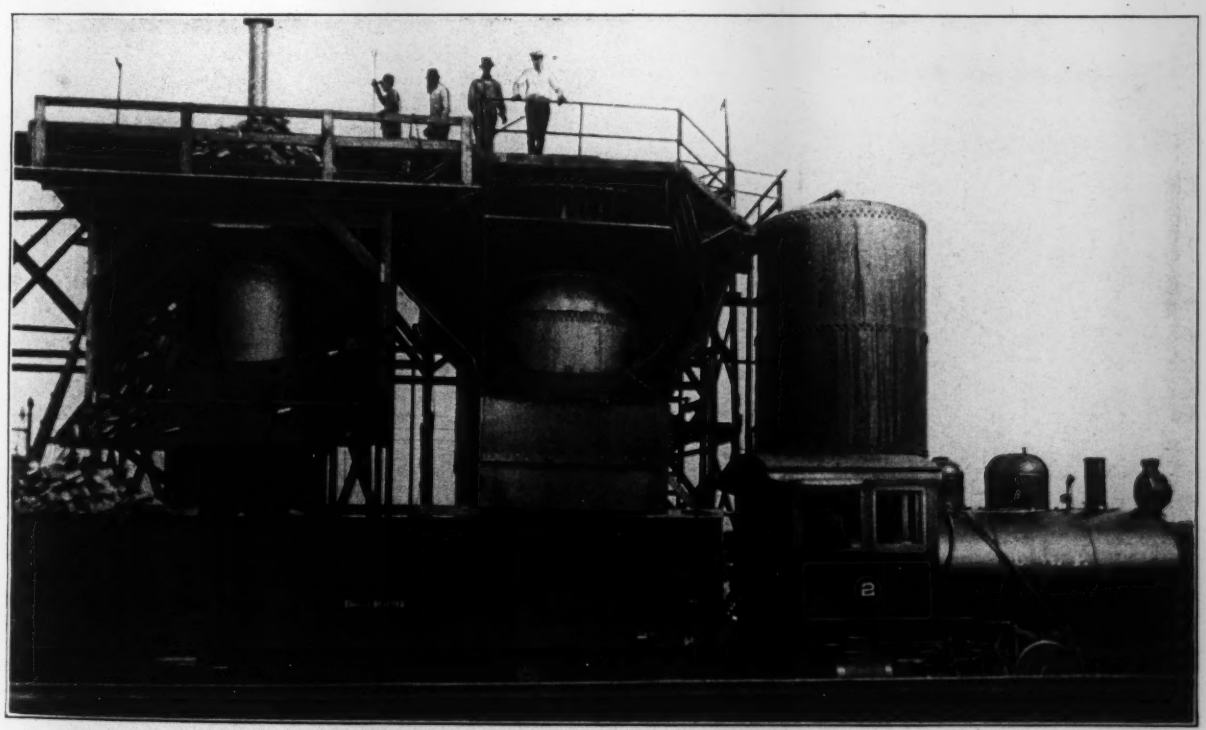


FIG. 1. CREOSOTED WOOD BLOCKS BEING EJECTED FROM VERTICAL TANKS AND LOADED INTO CARS DRAWN BY FIRELESS LOCOMOTIVE

the boiler being charged about seven times every 24 hr. at the main boiler, at 150 lb. pressure. The maintenance cost of this type of switching engine is low, and its use is very satisfactory in a treating-plant yard. Its tractive effort is fully equal to that of the usual type. Although it weighs only 22 tons, it has pulled as many as 12 loaded gondola cars at a time.

Qualifications for Membership in Engineering Societies

The principal requirements for admission to membership in each of the four great national engineering societies are set forth in the accompanying table, which has been compiled by Calvin W. Rice, secretary of the American Society of Mechanical Engineers. It will be seen that the requirements as to age and experience for full membership are about the same in the Mechanical, Civil and Electrical Engineers—namely, 10 years' experience and 30 to 32 years of age. Each of these societies has also a grade to which are admitted men of 25 to 32 years, who have had 5 to 6 years' experience, and also a grade for beginners in the profession.

These three societies have copied the practice of the British engineering societies, of which the Institution of Civil Engineers is the oldest, of grading those belonging to the society according to their age, experience and professional standing. The American Institute of Mining Engineers, on the other hand, was founded by men who had received their technical training in the famous mining schools of Germany; and it has always followed the practice of the great German Society of Engineers, which admits all members in one grade and makes no attempt to rate them according to their professional qualifications. One of the arguments in favor of this plan is that any grading of the members of a technical society is at best very imperfect. There are many engineers in the associate-member grade of the societies who are the superiors in professional ability of some of the men who hold full membership.

REQUIREMENTS FOR ADMISSION TO MEMBERSHIP IN VARIOUS GRADES IN FOUR NATIONAL ENGINEERING SOCIETIES

	Mechanical Member	Electrical Fellow	Civil Member	Mining Member
Age	32 yr.	32 yr.	30 yr.	All professional mining engineers, geologists, metallurgists or chemists and all persons actively engaged in these branches of engineering
Design and direct, as principal, important engineering work	Yes	Yes	Yes	
Practice of profession	10 yr.	10 yr.	10 yr.	
Responsible charge	5 yr.	3 yr.	5 yr.	
No. of references required	5	5	5	3
	Assoc. Mem.	Member	Assoc. Mem.	Associate
Age	27 yr.	27 yr.	25 yr.	All persons desirous of joining acceptable to the Board of Directors
Design or have charge of work, under direction	No	Yes	No	
Practice of profession	6 yr.	5 yr.	6 yr.	
In charge of work	1 yr.		1 yr.	
No. of references	5	4	5	3
	Junior	Associate	Junior	
Age	21-29 yr.	21 yr.	18-31 yr.	No grade
Practice	2 yr.		2 yr.	
Graduation from engineering school equivalent to practice	2 yr.	1 yr.	2 yr.	
No. of references	3	3	5	
Total membership	6,573	7,609	7,871	5,224

Shore Paving at Seaside Resort

During the summers of 1914 and 1915 the Metropolitan Park Commission of Massachusetts has been constructing shore protection at Revere Beach, an amusement resort only a few miles north of Boston, where an existing timber bulkhead had not prevented severe erosion of the foreshore and battering of the bulkhead by winter storms.

NEWLY BUILT SHORE-PROTECTION WORKS AT REVERE BEACH, MASS.

The protective structure is in the form of steps, as shown in the accompanying illustration. It is built of 1:2:4 concrete reinforced with ½-in. square rods and has a granolithic finish. Besides providing the desired protection to the shore, it furnishes seats for bathers and spectators who go there in enormous numbers on hot summer days, and it does not interfere with easy access to the beach from the boulevard, as a wall would.

Proper Use of Rams for Farm Water-Supplies

By W. G. Kirchoffer*

Many attempts have been made in the past to use hydraulic rams with flowing wells and springs, to secure water under sufficient pressure to supply farm buildings. Some installations have been more or less successful, but others have been failures, owing to the use of rams not of proper size or type. In the following paragraphs are

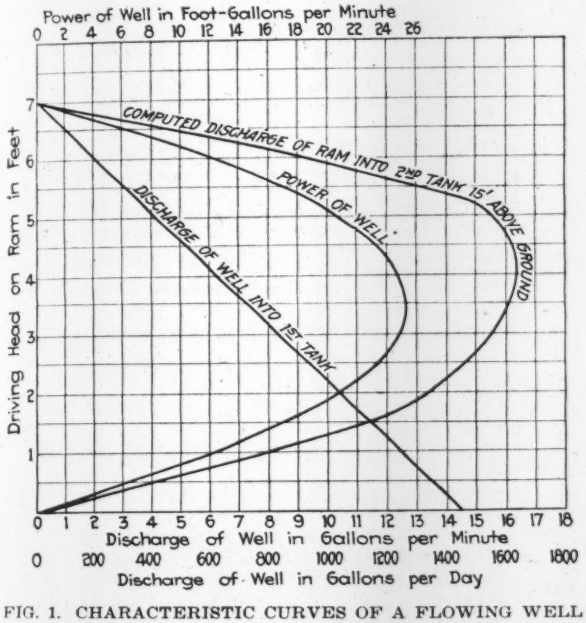

Power of Well in Foot-Gallons per Minute
0 2 4 6 8 10 12 14 16 18 20 22 24 26
COMPUTED DISCHARGE OF RAM INTO 2ND TANK 15' ABOVE GROUND
POWER OF WELL
DISCHARGE OF WELL INTO 1ST TANK
Driving Head on Ram in Feet
1 2 3 4 5 6 7
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
Discharge of Well in Gallons per Minute
0 200 400 600 800 1000 1200 1400 1600 1800
Discharge of Well in Gallons per Day

FIG. 1. CHARACTERISTIC CURVES OF A FLOWING WELL

set forth the simple hydraulics necessary for successful treatment of such problems.

We often hear the remark that a certain well will flow a certain size of stream, or so many gallons per day, at a given height above ground, but no other information regarding the characteristics of the well is known. Properly to understand the workings or characteristics for a given well, one should know the maximum height to which water will rise in a confined pipe above ground, or above a given elevation, where a ram might be located. The flow of the well should also be measured at several points above the given elevation. From such data it is an easy matter to plot a diagram such as is shown in Fig. 1 and marked "Discharge of Well Into First Tank."

In this particular instance the discharge curve happens to be a straight line. The equation of flow of this well is $G = 14.50 - 2.07\ H$, in which H is head in feet and G is the gallons per minute. The natural inference would be to think that, if the ram was placed at the ground level, we would have a head of 5 ft. and a flow of about 10½ gal. per min.; but such is not the case. If the well is allowed to flow freely at any given level, the head above that point is all consumed in discharging the water through pipe. The only head avail-

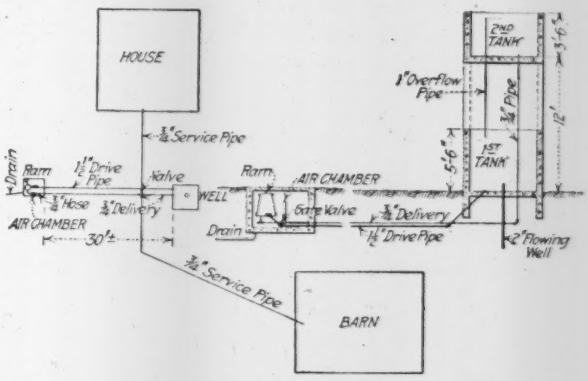


FIG. 2. PLAN AND ELEVATION OF A WISCONSIN FARM WATER-WORKS

able is the distance between this point of flow to the ground level, or to the bottom of the ditch—which in this case is 2 ft. below the ground level.

To arrive at the point where the power of the well is the greatest, so as to make the most efficient installation and secure the greatest discharge of the ram, it is only necessary to plot up the curve denoted as power of the well, which represents the product of feet of head and gallons per minute, or what is commonly called "foot-gallons per minute" (F). The equation of this curve is $F = 14.50\ H - 2.07\ H^2$. In this case H is the distance from the given elevation to the point of discharge.

The curve, as drawn, shows the power of the well for all points from 2 ft. below the ground to a point 5 ft. above ground, total head being 7 ft. The curve for this case shows that the greatest power of the well is obtained when the head is 3½ ft., or one-half the total head, and the discharge is 7½ gal. We should therefore select a ram that will work most efficiently when the head is 3½ ft. and the discharge is 7 gal. per min. The maximum power will be obtained at the middle point of the head on all wells whose discharge curve is a straight line.

Illustration of a Farm Supply

At the Wortham farm, near New London, Wis., there was a 2-in. artesian well 70 ft. deep. This well had the characteristics shown in Fig. 1. The total head of the well was determined by extending the well casing to a sufficient height so that the water would not flow over it, and the level of the water in the pipe was noted. Then the flow was measured in a 3-gal. pail at different elevations, which were noted. From these data the discharge line was plotted, and from this the power curve was derived. At this location the ground is level between the house and the barn, so that a head of 15 ft. above the ground surface was considered sufficient to supply the house and barn with the necessary water.

A concrete tank, about 6 ft. square by 5 ft. 6 in. high, was constructed over the well, and the well casing was cut off a few inches above the bottom. As the well was only a few feet from the house, it was decided to build a milkroom and cold-storage space about and over the

*Sanitary and Hydraulic Engineer, 31 Vroman Building, Madison, Wis.

tank. The upper tank, or the one into which the ram was to discharge, was located directly over the lower tank, so as to provide a cold-storage place between the two tanks and for the further reason that any surplus water pumped by the ram could overflow into the first tank and thereby help to keep up the driving head of the ram. By adjusting the operation of the ram so that it would take water from the lower tank at the rate of 7 gal. per min.—or what perhaps is easier, so that the head remains constant at 3½ ft.—a maximum amount of water will be pumped into the upper tank. The curve marked "Computed Discharge of Ram Into Second Tank," in Fig. 1, shows the amount of water in gallons per day that the ram will discharge at the different driving heads.

Objections to Rams Overcome

One objection to rams that has been made is that they are noisy, and if connected directly with a pipe leading into a house, the continual pounding becomes a nuisance. In the installation at the Wortham farm this objection was overcome by inserting between the ram and the discharge pipe a piece of rubber hose of diameter equal to that of the discharge pipe and by the use of an additional air chamber on the discharge pipe between the hose and the ram. The additional cushion furnished by the air chamber and the flexibility of the hose completely destroyed the propagation of the sound to the discharge tank. One could place an ear in contact with the discharge pipe as it passed through the milk house to the tank, and no sound could be detected.

Many rams will stop if the discharge head is suddenly increased, as it would be if pumping directly to a delivery pipe and the valve on it was partly closed suddenly. Also if the relief valve on some rams is purposely closed by hand, they will not automatically start again. However, the ram installed at the Wortham farm was adjusted so closely that it would start regardless of these conditions.

A plant similar to the one just described could be installed where the supply was from a spring, as well as from an artesian well. The same data should be collected and an analysis made of the problems, the same as for a well. High-efficiency rams will raise water where the ratio of driving head to discharge head is as great as 1 to 30, so that a head of 2 ft. with large discharge may often be of value.

Field for Automatic Pumps

Where the supply of water from the well or spring is relatively large as compared to the demand, an automatic pump could be used (double-ended reciprocating types like the Laursen machine). The particular advantage in the use of an automatic pump is that the elevated storage tank can be dispensed with and the discharge piped to the point of delivery. Whenever the faucet is opened, the pump starts to deliver water; and it automatically stops when the discharge is closed. These pumps can be used where the head and quantity of spring or well water might not be for house use or where there might be a head of water on a small stream sufficient to supply the power. These pumps can be operated safely where a polluted water is used for power, without danger of contaminating the pure water that is used for domestic purposes. They are also being used to supply water to railroads and factories from dams on small streams, where the purity of the water is not important.

Forms for Reporting Pavement History and Traffic

The forms shown herewith were presented at the last annual meeting of the Canadian Society of Civil Engineers by the Committee on Roads and Pavements.

HISTORY OF PAVEMENT

Municipality.....Kind of pavement.....Year laid.....
Engineer in charge.....Present address.....
Name of street or road.....From.....To.....
Total width or road allowance.....Bearing of street.....
Length of pavement.....Width.....Area in sq.yd.....
Cost per sq.yd., exclusive of subgrade, curb and gutter.....
Subgrade—Cost per sq.yd. of pavement.....
Subsoil.....crowned or flat.....watered or dry.....
What provision for drainage of subgrade.....
Were sewers, water mains, etc., laid before construction of pavement ..... and .....
When.....Location with respect to pavement.....
Foundation—Cost per sq.yd.....
Kind..... Materials..... Thickness..... Proportions.....
Binder Course or Cushion—Cost per sq.yd.....
Kind..... Materials..... Thickness..... Proportions.....
Surface Course—Cost per sq.yd.....
Kind..... Materials..... Thickness..... Proportions.....
Curb—Cost per lin.ft.....
Kind.... Width.... Depth.... Materials.... Proportions.....
Gutter—Cost per lin.ft.....
Kind... Width... Thickness... Materials.... Proportions.....
Details of catch basin.....
Grade of pavement, minimum.....maximum.....ordinary.....
Crown of pavement, minimum.....maximum.....
Character of street, business, industrial, etc.....
Average daily traffic in terms of traffic schedule before and after improvement .....
Summer temperature, maximum....minimum....average.....
Winter temperature, maximum.....minimum....average.....
Annual rainfall, in.....Annual snowfall, in.....
Depth of frost line.....
Weather conditions during laying of pavement.....
Remarks .....
Dimensioned sketch of cross-section of pavement, photograph also desirable.....
Municipality.....Street or road.....From.....To.....
Kindly state below all details regarding the following:
- Form in which wear of pavement was shown from year to year since construction.
- Methods of repair.
- Methods of maintenance.
- Cost of maintenance per sq.yd. of pavement.
- Cost per sq.yd. of repairs.
- Average daily traffic.
- Any additional information that will be of value.

CAN. SOC. C.E. FORM OF TRAFFIC CENSUS

Kind of Traffic	7-8 a.m.	8-9 a.m.	9-10 a.m.	10-11 a.m.	11-12 a.m.	12-1 p.m.	1-2 p.m.	2-3 p.m.	3-4 p.m.	4-5 p.m.	5-6 p.m.	6-7 p.m.	Totals
One-horse light vehicles													
One-horse heavy vehicles													
Empty													
Loaded													
Two-horse light vehicles													
Empty													
Loaded													
Two-horse heavy vehicles													
Empty													
Loaded													
Run-abouts													
Light touring cars													
Empty													
Loaded													
Heavy touring cars													
Empty													
Loaded													
Light motor trucks													
Empty													
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Loaded													
Weather													

Monolithic Concrete Curb and Pavement in Arcade, N. Y.

By E. J. Doyle*

SYNOPSIS—Pavement has two courses. Concrete placed in 25-ft. sections separated by expansion joints. Details of curb forms. Amount of water used in concrete different in dry than in wet weather. Work proceeded at rate of six sections, or 70 to 90 cu.yd. per day. Costs.

A concrete pavement was constructed for the village of Arcade, N. Y., in 1915 that has attracted considerable attention because of some unusual features of construction that were employed. The distinctive features were the construction of curbs and pavement in monolithic units, reinforcing each unit with bars to prevent temperature cracks and using a top course of concrete containing traprock instead of gravel, as in the lower course.

Constructing the curb and pavement as a unit reduced the formwork of curbs to a minimum, centralized all concreting and shortened the period of construction. Under the ordinary method a full organization must be employed, and the work is a duplication of the force and operations required in laying down the pavement.

The cross-section chosen is shown in Fig. 1. It called for two courses of concrete, the bottom course being gravel concrete and the top of stone. As local gravel of good quality could be obtained at low cost, its use effected a considerable saving.

Description of the Pavement

The street had a 22-ft. roadway, with tree lawns on each side. The maximum grade was 1.8%, the minimum, 0.34%. The bottom course was a 1:3:6 (screened gravel) mix. Stone was used in the top course and was graded from chips to 1-in. pieces. It was tested for voids, which were found to be between 22 and 28% of the bulk. Tests of a 1:1½:3 mix indicated that the mortar was too unstable when sufficient water had been added to bring it to the desired degree of fluidity. Accordingly a 1:1:3 mix was used for the top-course concrete.

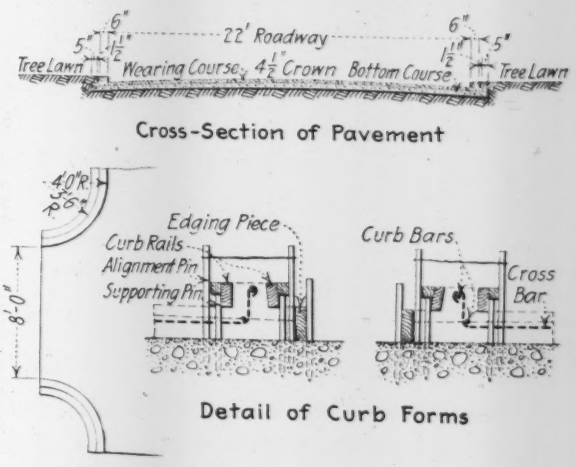


FIG. 1. DESIGN AND CONSTRUCTION DETAILS OF THE ARCADE (N. Y.) CONCRETE PAVEMENT

The bottom course was 4 in. thick at the sides and 5 in. at the center. The top course had a uniform thickness of 2 in. The concrete was placed in 25-ft. sections, the sections being separated by a ⅜-in. prepared

*Consulting Engineer, 83 State St., Albany, N. Y.

FIG. 2. CONSTRUCTING CURB AND PAVEMENT IN MONOLITHIC UNITS

asphaltic expansion strip. The curbs, which rose 4½ in. above the pavement, were reinforced with two ½-in. bars. The pavement was reinforced with crossbars of the same size, spaced 4 ft. c. to c. The crossbars were looped at each end and bent up in an L-shape in the curbs. One curb bar was passed through the loop, and the other was placed on the inside of the L, to give the crossbar an anchorage.

The curbs had no expansion joints except those at the end of a section, which were the same as the roadway

A plank bulkhead that conformed to the surface of the subgrade and finished pavement was secured in place with pins. A trussed templet, used in forming the cross-contour of the top and bottom courses of concrete, was made of light pine boards, so that its weight would not depress the curb rails and deform the contour of its templet edge. The contour of the bottom course of concrete was obtained by floating under the templet with a float that had a spacing block attached to the top of its blade. The street-intersection and house-drive

FIG. 3. THE COMPLETED TWO-COURSE PAVEMENT ON PARK ST., ARCADE, N. Y.

joints. The roadway was crowned 4½ in., and a hyperbolic profile was adopted to allow for sloughing of the fresh concrete from the center and the consequent rising of the quarter-points. Cross-sections of the hardened pavement showed that the sloughing was of such extent that the profile of the final cross-section conformed very nearly to the arc of a circle.

The Curb Forms and How They Were Placed

The curb forms consisted of two curb rails and an edging plank. The rails were supported on iron pins, which held them 6 in. above the subgrade and on the plane of the pavement surface. The edging plank was set on the subgrade, outside the rails, to form a 5-in. heel. The rails were made up of 6-in. planks sawed to 4½ in., with a 1½-in. piece spiked to the outside of the rail, at the top, to give the rail an inverted L-shape. The rails were fitted with dowels at the ends, to hold them in alignment without nailing. The supporting pins were spaced about 4 ft. c. to c., and the forms were held in alignment by pins driven along their sides at short intervals and wired across. The supporting pins were of round iron, to facilitate their removal.

In setting the curb forms batten boards were set on stakes opposite the grade and alignment stakes, a line was drawn, the supporting pins were put in position and driven to about ½ in. above their final grade, and the rails were placed with the long arm of the L in a vertical position inside the pins and the outstanding short arm resting on the pins. Alignment pins were then driven and wired. No effort was made to set the edging plank on an exact grade, as the heel of the curb was covered when completed.

Before concreting a section, the curb rails were brought to true grade by tamping them over their pin supports.

radii were each in a built-up unit and were set and secured in the same manner as the straight forms. Bulkheads for the house drives were made with a 1-in. crown.

The concrete was mixed by a ½-yd. Koehring mixer with bucket loader and crane delivery. The sand and gravel were screened by a portable rotary screen driven by a gasoline motor. The screen had a capacity of 90 cu.yd. in a 10 hr. working day. The gravel was conveyed from bed to screen by two teams on drag scrapers, and the screen was moved along as the bed was worked.

Only two men were required to set and reset the curb forms. The concreting gang was made up of a foreman; a fireman; an engineer; a bucket-hoist man; a crane operator; two men on the cement; seven on sand, gravel and stone; three manipulating concrete on the grade and filling curb forms; one on the first float over; one pavement finisher, and one curb finisher. One team and an extra man were required to haul cement to the mixer.

The subgrade was rolled after a light shower or, in dry weather, after sprinkling. This procedure resulted in a hard surface that was damaged but little by hauling supplies over it. All materials except the cement were delivered a considerable distance ahead of the mixer. The cement was delivered as required and in most part was taken directly from the cars. The sand, gravel and crushed stone were dumped in rows on the subgrade, each class of material being piled in a separate row.

Each batch of concrete was mixed about 50 sec. The bottom-course concrete was mixed with sufficient water to make it flow sluggishly while being manipulated on the subgrade—a consistency that made it possible to do most of the spreading with floats and long-handled hoes. The top-course concrete was made plastic. No fixed

quantity of water was used in either the top or the bottom course, as the quantity that was required varied greatly in dry and showery weather.

Before concreting a section, the form-setter graded up the curb forms, and the curb finisher placed the reinforcing bars. The bulkhead was placed and lined up with an expansion strip. The bottom course of concrete was laid down to the half-point of the section and floated off. It was followed with the top course to the third point. The other half-section of the bottom course was then laid down and the section completed with the top course. This procedure was followed so that the workmen could manipulate the courses without standing in the concrete that had been floated off. The floating was done with steel floats that had been fitted with long bamboo handles.

After a half-section of the bottom course had been laid down and floated off ready to receive the top course, part of its water had run off. This hardened the concrete to such an extent that it did not intermix with the top course, except that the mortar coat which had been floated to the surface embedded the stone of the top-course concrete. The top of the curb was filled with top-course concrete, which was shoveled into the forms as it was dumped from the bucket. The curbs were given no mortar coat, and a natural mortar finish was obtained by troweling them out while the concrete was fresh.

The top course was floated over three times, the final floating being done as soon as the final set had started. In warm weather the fresh concrete was sprinkled frequently, and in showery weather it was covered with tar paper laid crosswise. The curb forms were removed the morning after the concrete had been placed. One man removed the forms of a day's run and cleaned and carried them ahead for resetting, in about 2 hr. The finisher drew the pins and grouted up the holes they left, as soon as the forms had been removed.

The fresh concrete was not covered with sand, but was sprinkled at frequent intervals for several days during dry warm weather. The pavement was opened to traffic in 12 to 14 days after placing. A normal day's run was five to six sections, which with house drives and curbs required from 70 to 90 cu.yd. of concrete. The regular working day was 10 hr. No section was started that could not be completed in a continuous run; and if a section was completed an hour before quitting time, as was often the case, the men were put to work grading tree lawns back of the mixer.

A record of costs indicated that the pavement, including excavation and grading, was constructed for about 80c. per sq.yd. and that the curbs, including radius curbs, cost 21c. per lin.ft. The items making up the cost of curb were: Forms, 3c.; setting forms 3c.; concrete, 10c.; reinforcement, 3c. and finishing, 2c.

Plans and specifications were prepared by, and the work was performed under the general supervision of, the writer, who was consulting engineer for Arcade. R. T. Spiegel was in immediate charge. The contractor was Frank M. Foote. The photographs were supplied by F. M. Traynor, of the Lehigh Portland Cement Co., and by the contractor.

January Rainfall in Mountains Near Los Angeles*

At the instigation of members of the American Society of Civil Engineers there have been established in southern California by the United States Weather Bureau 11 rainfall stations, located at various elevations in the mountainous regions. These stations were established with standard equipment by Dr. Ford Carpenter, local observer. During the heavy rains of January, 1916, some of these stations were destroyed, so only the data herewith are available.

INCHES OF DAILY RAINFALL IN JANUARY, 1916, AT MOUNTAIN STATIONS IN LOS ANGELES COUNTY, CALIFORNIA

Station*	Elevation	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Clearwater power station	2,350	....	1.55	0.03	0.08	....	0.02	....	....	0.98	0 60	....	....	....	....	0 61	
Camp Bonita	2,000	....	1.78	0.24	0.18	0.36	0.17	....	....	1.20	0.81	1.00	....	....	....	0 88	0.05
Iron Forks, San Gabriel River	3,500	1.50	0.50	0.45	0.31	....	....	....	1.43	....	1.80	....	....	....	....	1 12	
Littlelands	1,800	0.01	1.20	0.80	0.15	0.30	0.10	....	....	0.75	0.35	0.56	....	....	0 05	0.80	0 01
		2.0†	trace†			trace†	0.10†			4.0†					2 0†		0 5†
Mt. Wilson	5,850	0.27	1.47	0.03	0.57	0.42	0.18	....	1.23	1.00	....	....	....	....	0.62	0.31	0 58
Neenach (Antelope) siphon	3,000	5.0†	0.25	0.20	0.12	....	....	....	....	0.04	0.07	0.7†	....	....	....	0 25	
Pacoima	1,570	....	0.60	0.40	0.26	0.55	0.09	....	....	0.96	0.45	0.15	....	....	....	0 20	

Station	Elevation	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	T. a
Clearwater power station	2,350	....	4.40	0.30	....	....	....	....	0.17	0.25	....	1.85	....	0.50	....	....	
Camp Bonita	2,000	5.18	8.90	1.60	0.20	....	....	....	....	0.76	0.05	1.54	3.32	....	0.52	....	28 74
Iron Forks, San Gabriel River	3,500	....	———	16.75	———	....	....	....	....	———	4.55	———	....	0.60	....	....	
Littlelands	1,800	4.08	3.38	0.50	0.15	....	trace	....	0.08	0.60	0.04	1.22	0.91	0.51	....	....	21 04
			1.5†	8.0†								6.0†	0.5†	5.0†			
Mt. Wilson	5,850	6.38	4.37	0.18	0.96	....	....	0.02	0.36	0.76	0.03	4.4	0.03	0 30	....	....	
Neenach (Antelope) siphon	3,000	1.40	0.90	0.25	....	....	....	0.04	....	....	....	0 12	....	....	0 09	....	3.70
Pacoima	1,570	4.50	3.05	0.57	0.15	....	....	0.10	....	0.75	0 05	1.36	1.09	....	0 40	....	

* All the stations are in Los Angeles County. † Snow.

At Los Angeles the total precipitation for January, 1916, was 13.3 in., as compared with a normal of 4.16 in. From Sept. 1 to Jan. 31 the total rainfall at Los Angeles was 17.17 in., contrasted with 8.05 in. normal. High daily rainfalls at Los Angeles during January since 1877 have been 7.83 in 1890, 7.72 in 1886, 6.03 in 1888 and 5.84 in 1895.

Development of the Foreign Trade of the United States is urged by the National Chamber of Commerce. A referendum vote has just been taken of commercial organizations throughout the country; 307 different organizations located in 42 states took part in the ballot. The vote was on the approval of the report of a committee that advocated increased appropriations and a strengthening of the staff of the United States consular service and the Bureau of Foreign and Domestic Commerce. Five new commercial attachés were recommended, special attention was given to the development of commerce in Latin America, and it was urged that the consular service be Americanized. On the detailed propositions in the ballot the vote was generally 850 to 875 in favor, with only from 8 to 40 in opposition. In this connection it is of interest to note that Dr. E. E. Pratt in his annual report declares that the chief difficulty in the development of foreign trade is the lack of competent educated men to conduct it. He states that his bureau finds it difficult to obtain men of education and good address who are good linguists and have a fair knowledge of commercial geography.

*From Roy A. Silent, Engineering offices of J. B. Lippincott, Central Building, Los Angeles, Calif.

Design of Rock-Fill Dams

SYNOPSIS—An account of the failure of the Lower Otay dam, near San Diego, Calif., on Jan. 27, with editorial comment thereon, published in "Engineering News" of Feb. 3 and 10, has brought forth discussion by four engineers of the principal features in the design of such dams. It is contended that they should be given a more ample cross-section than has generally been adopted and should not be expected to withstand overflow. Comment on this discussion is made in the editorial columns of this issue.

Rock-Fill Dams Tested by the Record of Experience

By Francis L. Sellew*

Engineering News of Feb. 3 contains an account of the destruction of the Lower Otay dam, near San Diego, also a description of the work, with some comment upon its design and construction, which leaves the impression that the sole cause of the disaster was structural weakness. The structure was weak, and your criticism is fully justified. Still, a familiarity with local conditions leads one to suggest that, had the managers of the works manifested a decent regard for its infirmities, the dam could have been saved and a needless accident avoided. No detailed analysis of the dam will be attempted here, and none is necessary, for a superficial examination will show its weakness with a full reservoir—a condition which should never have been allowed to occur.

Assume the cross-section which you publish, having a top width of 16 ft., side slopes of 1½ : 1, height 130 ft., with steel diaphragm located on the line of the top of the water slope, allowing free access of the water pressure to that plane, leaving the rock-fill below the diaphragm the sole dependence for stability, as you suggest. With the water-level even with the top of the dam the normal water pressure on 1 lin.ft. was $62.5 \times 130 \times 65 = 528{,}000$ lb., or 264 tons. Opposed to this would be a rock-fill of 14,750 cu.ft., or 550 cu.yd. In the absence of specific information let us assume that the rock weighed, solid, 150 lb. per cu.ft., or 4,050 lb. per yd., and when broken, contained 40% voids, giving, say, 2,500 lb. as the weight of a cubic yard in place. Then the opposing force would be $550 \times 1\frac{1}{4} = 690$ tons and the coefficient against sliding, 690 divided by 264, which is 2.6.

COEFFICIENT AGAINST SLIDING

Location of Dam	With Reservoir Full	With Reservoir 5 Ft. Below Top	Remarks
Lower Otay, slopes 1½ : 1	2.6	2.85	As designed
Lower Otay, slopes 1 : 1	1.85	2.04	As built, 1897; failed, 1916
Escondido, Calif. ...	3.25	3.50	Built, 1895
Castlewood, Colo. ..	2.1	...	Built, 1890; rock-fill with masonry faces
East Cañon Creek, Utah	4.0	...	Built, 1899; 68 ft. high; enlarged, 1900, to 93 ft.
Pecos River, New Mexico	3.35	...	Built, 1890; failed, 1893; rebuilt, failed, 1894; rebuilt
Walnut Grove, Ariz.	2.60	...	Built, 1887; failed, 1890

The spillway floor was at El. 124, or about 6 ft. below the crest of the dam; and had the water been held at that level, the normal pressure would have been 240 tons with the same opposing force, giving a coefficient of about 2.9. However, as you point out, the structure probably was not built to the design, and the slopes were more nearly 1 : 1.

Under such conditions with a full reservoir the normal pressure would be, as before, 264 tons; but the opposing force would reduce to 390 cu.yd., or 490 tons, and the coefficient against sliding to $\frac{490}{264}$, or 1.85. Had the water been held at spillway level, the coefficient would have been $\frac{490}{240}$, or 2.04.

This analysis brings up the question of a safe coefficient against sliding for a rock-fill dam. In masonry-dam design a common value for this coefficient is 2, and for earth dams the best practice gives 10. It is apparent that the safe figure for a rock-fill will lie between these two; and as a guide for determining its proper value the accompanying table has been prepared from an examination of several structures, of which some have failed and others are still standing.

The Escondido dam, located a short distance from San Diego, has a maximum height of 76 ft.; a top width of 10 ft.; water slope, ½ : 1; downstream slope, 1 : 1 for the top half and 1¼ below; total bottom width, 140 ft. In the calculations the vertical component of the water pressure was added to the weight of the fill to make up the resisting force. The upstream slope of the dam has a plank face, which is assumed to be impervious. The entire weight of the fill is therefore available to resist the water pressure. This dam has withstood successfully all floods since 1895.

The Castlewood dam, Colorado, was built in 1890, the rockfill having a top width of 8 ft.; water slope, 1 : 10; downstream slope, 1 : 1; total bottom width, 85 ft. The water face is of rubble masonry 4 ft. thick, and the lower slope is built in 2-ft. steps with large stones in cement mortar. There was considerable leakage, and an earth-fill was added on the upper side with 8 ft. top width and 3 : 1 slope. This addition was merely to reduce leakage, and in determining the coefficient against sliding the rock-fill was considered the sole support. No allowance was made for the rock-fill acting as a retaining wall for a part of the earth-fill.

The East Cañon Creek rock-fill dam, Utah, was originally built 68 ft. high in 1899. It was enlarged in 1900, its principal dimensions being: Height, 93 ft.; crest width, 10 ft.; water face, vertical and covered with boiler plate; downstream slope, 2 : 1. The bottom width is about 250 ft. Boiler plate is connected to a concrete cutoff, which extends to bedrock. In the calculations the upstream face was assumed impervious and the water pressure resisted by the entire rock-fill.

The Pecos River dam was built in 1889-90 near Eddy, N. M. Its height is 48 ft.; crest, 10 ft.; water face, ½ : 1; downstream face, 1½ : 1; bottom width, 106

*Consulting Engineer, La Jolla, Calif.

ft. The foregoing figures are the dimensions of the rock-fill; but to insure water-tightness, an earth embankment was added on the upper side, the top width being 10 ft. and the slope 3:1. The amount of earth so placed was 100,000 cu.yd. In figuring the factor of safety the opposing force has been considered as the rock-fill plus the vertical component of the water pressure on the inclined upstream face. This dam was overtopped by a flood in 1893 and washed out. It was immediately rebuilt with additional spillway capacity and the rock-fill given 5 ft. additional elevation. Referring to this dam, Wegmann says: "The type of rock-fill dam described above appears to be perfectly safe if an ample wasteway is provided."

However, the dam was again destroyed in 1894, the water this time forcing its way through, but not passing over, the top. Its failure was stated to be due to burrowing animals or faulty construction where the dam connected with the bank. The dam was again rebuilt with increase in the earth-fill and the addition of a concrete core wall. The calculations previously given for safety against sliding relate to the original cross-section.

The Walnut Grove dam, in Northern Arizona, was built in 1887 and failed in 1890. The structure had a top width of 10 ft.; maximum height, 110 ft.; water slope, 1 horizontal to 2.3 vertical; downstream slope, 1 horizontal to 1.55 vertical for the top half, the lower portion being increased, the dam having a bottom width of about 175 ft. Probably the steepness of the downstream slope had much to do with the failure of this structure, although its sliding coefficient is less than it should be.

An examination of the table shows that the Walnut Grove dam had a coefficient against sliding, with full reservoir, of 2.6, or about 50% greater than Lower Otay, while the Pecos River rock-fill, which went out in 1893 and again in 1894, had a coefficient 80% greater. If allowance is made for retaining-wall action at Lower Otay and the weakening of the section on account of saturation from leakage, the discrepancy between the coefficients is still greater; but comparing the values as given in the table, these two failures—one occurring seven years and the other four years before the Lower Otay was built—should have caused the designers of the latter structure to "Stop, look and listen."

Eliminating the Castlewood dam, which by reason of its masonry faces is rather a modified rock-fill, the table shows that the successful rock-fill dams had in the case of Escondido a coefficient of 3.25 and at East Cañon Creek, 4. The Escondido dam was built subsequent to the failure at Pecos River and at Walnut Grove, but prior to the construction of the Lower Otay. It would have been a fair guide to follow, but probably was so near that it was overlooked.

It appears from the data herewith submitted, after making allowance for all factors, certain and uncertain, that a rock-fill dam should have a coefficient against sliding of at least 3½, and 4 would be better. If this assumption is correct, then the maximum height to which the Lower Otay reservoir should have been filled was 93 ft. and not over 130, as it was at the time of failure; even at spillway level, 6 ft. below the top of the structure, the factor was 2.04.

The facts here presented are matters of record, readily obtainable. It is therefore pertinent to ask why the reservoir was allowed to fill, when such a condition was almost certain to cause disaster. The structure was so weak under maximum conditions that the filling of the reservoir was as great an error as the design of the dam.

Action of Water on Rock Fills

By Ralph Bennett*

Referring to your recent editorial on the stability of rock-fill dams, you appear to assume that the downstream face of a rock-fill dam, properly surfaced with stone blocks, can be used as a spillway for extensive flood flows. I do not believe that either the correct theory or the successful practice with structures of this character will permit of any flow whatever being discharged over the back of such a structure.

The stability of any dam, properly founded, is dependent on its resistance as a mass to overturning or on its stability as a framed structure, but in either case the materials must be protected against progressive destruction, disintegration or removal.

A rock-fill dam cannot be considered to have any features of a structural nature. Even when carefully derrick-placed on the upstream slope, the downstream face is random dumping. Such a rock pile contains many crevices and is likely to be composed to a considerable extent of angular fragments resting on corners so small that the material is very highly loaded at the points of contact. Depending on the shape of the fragments and the manner of placing, the slopes will be internally stable at a slope that may be averaged at 1 on 1. With this slope, superimposed vertical load will cause more or less extensive slips until the slopes have adjusted themselves to the new conditions. A rock pile with a materially flatter face, say 1 vertical on 1½ horizontal, is reasonably free from such slips. This flatter slope is quite generally assumed as amply safe for rock-fill dams, and occasionally much steeper faces are used with favorable rock and some hand placing; but the enormous masses required on flat slopes require that the steepest safe face be used.

When loaded at an angle perpendicular to the water face with a water load, as in the case of full reservoir, the internal stresses in the rock-fill dam may be supposed to travel along lines that are the resultant of the water pressure and the dead weight of the dam, as in the case of a masonry dam. But the distribution of the stress will be much less even in the loose rock than it will be in a concrete mass. The semifluidity of the pile will transfer portions of the loading to the sides of the pressure lines and so toward the unrestrained outer face. A condition may then arise under which a bulging and failure of the outer or downstream face of a rock-fill may occur, owing to pressure from the upstream face. This condition is analogous to the transfer of pressures in a grain bin. There appear to be few or no experiments on this most interesting and important matter.

Under the conditions most frequently met with there is a considerable amount of earth, sand and spall embodied in the rock-fills. As this material may be more slippery than the rock of the body of the fill, it may increase the chance of failures due to internal stresses. This result will particularly be the case if the mass is wet.

*Consulting Engineer, Washington Building, Los Angeles, Calif.

It may safely be said that a rock-fill dam designed to suit the materials employed and sustaining a full reservoir will have a certain amount of load on every particle of its material and that minor slips and subsidences may be expected, if the structure is too light, well before full reservoir is reached.

Now, if water in considerable quantities is allowed to flow over the crest and down the lower face, several additional hazards are at once in evidence: There is considerable shock and vibration; the surface rock, even if massive and laid in courses, is called on to resist a very strong loosening and tearing action due to the mass and high velocity of the water; the fill, being less porous on the surface than in the body, will fill with water entering the crevices at all heights and be prevented from venting freely by the surface water; this entrained water will soften and lubricate the sand and dust present and will wash them away by slow attrition, even if loaded.

This water will also to a greater or less extent submerge the rock of the fill, which is held in place by its weight, and by reducing its effective weight reduce the total resistance of the dam. If already well loaded, the failure due to slips and slides from these causes may occur before that due to surface erosion.

When the rock-fill dam has had added to it all the crest width, paving and drains that would be essential to enable it to handle even a thin stream over its crest, its cost will so far exceed that of a single or multiple-arch dam for the same service that the only argument for its use—cheapness—will vanish.

If the conditions render the use of a long shallow spillway difficult or undesirable, it is possible to install siphon spillways, which will immensely increase the discharge per foot of crest and which will operate with great certainty on a head as low as 4 in. Some types of automatic flashboards are also available, which can be relied on. But in any case an ample length of free spillway crest is the safest insurance against overtopping failure in earth- and rock-fill dams.

Stresses in Arch Dams

By L. Jorgensen*

It may be that the Lower Otay dam was not held in such high repute by the engineering profession, but its failure was evidently due to washing out of the structure by the water overtopping its crest; and no matter how liberally the section had been proportioned, the result would probably have been the same. The only real protection would have been a larger spillway or a concrete structure. The lining of the crest and downstream face would undoubtedly be more expensive than a larger spillway and would not offer the same protection. In the writer's opinion well-constructed rock-fill dams are perfectly safe structures, if provided with sufficient spillway capacity; but for efficiency of material the water-tight cutoff should always be on the upstream face and not in the shape of a core wall.

When the first reports of the dam failure came out in the newspapers, its was not stated whether it was the lower or the upper dam that had failed. The writer, knowing that the upper dam was an arch dam and being especially interested in arch dams, started to figure out how the upper dam could have failed. He knew it was the highest-stressed arch dam in existence. As can be seen from the drawing and photographs on p. 288, *Engineering News*, Feb. 10, 1916, the arch is very flat. If this dam had failed, it would hardly have been fair to regard it as an arch-dam failure, although if it were not for its arch action, this dam would not now be standing. So far as the writer is aware, no arch dam has ever failed.

The stresses in the Upper Otay dam are difficult to ascertain. At lower elevations the dam has practically no curvature, and the load therefore must, to a great extent, be carried on the horizontal (curved) beams and through shear action to the rock sides and bottom. In the middle portion there will be some arch action, some horizontal curved-beam action, and some of the load will undoubtedly be transferred to both upper and lower elevations through vertical-beam action. The upper portion of the dam will then carry more than its corresponding water load on the arch. The upper portion of the arch is constructed with a rich mixture of concrete, which adds greatly to the safety of this structure. The reinforcement is present in such small quantities that it is negligible. Any structure that has withstood such a test without showing any signs of weakness is likely to have its "ultimate fate long delayed," as you say.

*Electric and Hydraulic Engineer, Chronicle Building, San Francisco, Calif.

Why Rock-Fill Dams Are Not Adapted To Resist Overflow

By Horace W. King*

In the editorial in *Engineering News* of Feb. 10, entitled "Is the Rock-Fill Dam a Safe Type of Dam?" three fundamental defects in the design of the Lower Otay dam are stated—namely, too small a cross-section, failure to pave the downstream slope and lack of a substantial crest to the dam. Without these defects, it is maintained, the dam would have withstood the flood of water that passed over it.

The writer is rather surprised at the position taken in this editorial, as he had not supposed that the rock-fill dam was ever considered as of the overflow type. In this respect it stands almost identically in the same class with the earth-fill dam. The rock-fill dam has an advantage over the earth dam in that failure is not so likely to result from seepage starting along the outlet works or at the junction with the banks of the stream, but it has no advantage in the regard mentioned. The writer cannot conceive of rocks weighing from one-half ton to three tons being so well laid that the terrific force of a large volume of water passing down the lower face of the dam will not dislodge them. The writer would like to inquire whether an overflow rock-fill dam is anywhere in existence.

In the case of the Lower Otay dam it appears that the structure withstood the pressure of the water safely until it was overtopped. In this respect it did all that any dam of this type could be expected to do. The real cause of the failure was insufficient spillway capacity. A separate spillway of capacity sufficient to discharge the maximum flood with the water surface in the reservoir at least 10 ft. below the crest of the dam is not too great a margin of safety.

*Professor of Hydraulic Engineering, University of Michigan.

Rock-fill dams are particularly suited to certain localities in the western United States, on streams flowing through rock cañons where suitable material for an earth dam is scarce and materials for masonry dams are prohibitive in cost. In such cases a rock-fill dam may provide the most economical and satisfactory solution of the problem. A spillway, detached from the structure, must always be provided for dams of this type. The spillway may, if topographic features are suitable, be located in a saddle remote from the dam, or it may be necessary to construct it in the sides of the cañon just above the dam. In the latter case the material excavated from the spillway may be deposited in the dam.

A substantial cutoff extending from an impervious foundation to the elevation of the maximum water surface is essential in a dam of this type, but rock of an indifferent character may be used in the body of the dam. Rock may be used for this purpose that would not be satisfactory for a masonry structure, which fact may lead to the choice of a rock-fill dam rather than a masonry dam for a particular locality.

In the writer's opinion a rock-fill dam, if properly designed, may be a perfectly satisfactory structure, but it will be safe only in case sufficient spillway capacity is provided to prevent water from passing over its crest. The three or four failures of rock-fill dams that have been recorded all appear to be traceable to one cause—failure to provide adequate spillway capacity.

Work of the Dallas Department of Public Health

EDITORIAL CORRESPONDENCE

Dallas, Tex., has a bureau of sanitation directly supervised by engineers. The city department of health was created by the city commission on May 5, 1915. It is theoretically under the general supervision of an advisory board of health (unpaid) of 13 members, appointed by the mayor, who is chairman. The law requires that 6 of the 13 members shall be practicing physicians. The board is composed of prominent physicians whose connection with it gives it considerable popular prestige.

The work of the department is divided into three bureaus or divisions—sanitation, care of the sick, hospital work—each in charge (theoretically) of a committee of the board of health. The bureau of sanitation is in charge of Charles Saville, a graduate in sanitary engineering of the Massachusetts Institute of Technology and formerly with Hering & Gregory, consulting engineers, New York City. The assistant chief of the bureau of sanitation, Henry W. Van Hovenberg, is also a graduate in sanitary engineering of the Massachusetts Institute of Technology. The heads of the other two bureaus are physicians.

Under the director of sanitation are two divisions—one of sanitation and public-health instruction and one of food and milk inspection and analysis. The division of vital statistics is under the health officer, who heads the bureau of caring for the sick. The collection and disposal of garbage and refuse and the cleaning of the streets are not a part of the work of the department of health, but are in charge of superintendents who report to the commissioner of streets and public property. However, the bureau of sanitation coöperates with these departments as well as with the water and sewer departments, and the director of sanitation is now engaged in a special study and investigation of garbage disposal.

The principal work of the bureau of sanitation is the inspection of food and milk supplies and the inspection and abolition of nuisances. Systematic inspections are made of all dairies and of all places where food is offered for sale. The condition of these places is rated on score cards that take into consideration the sanitary conditions of the building and equipment, the character and condition of the foodstuffs, the health and cleanliness of employees, toilet and washing facilities. These items are rated according to their relative importance, so as to make 100 a perfect score. The scores are furnished to housewives and others on request and are a stimulant to general cleanliness on the part of all who handle foodstuffs. Inspectors of the bureau of sanitation also condemn

SANITARY INSPECTION – CITY OF DALLAS – HEALTH DEPARTMENT

Street ..... No. ..... Date .....

Owner / Agent ..... Occupant ..... White / Colored / Mexican

Number of occupants, total ..... Children under 5 years .....

WATER SUPPLY

City Supply—*Yes, No.* In House—*Yes, No.* On Premises—*Yes, No.*

Other Supply—*Yes, No.* Describe .....

Possible Pollution of Supply—*Yes, No* .....

SEWAGE

Sewer Connection—*Yes, No.* In House.—*Yes, No.* Condition—*Good, Bad*

Dry Closet—*Yes, No.* Used by Family—*Yes. No.* Used by Servant—*Yes No* Condition—*Good, Bad*

GENERAL

Milk supplied by Dairy—Occupant—Neighbor—Store

Roomers, Boarders—*Yes. No.*

Condition of Premises—*Good, Bad*

Animals (No.): Dogs ..... Cows ..... Horses ..... Mules ..... Hogs ..... Chickens ..... Cats .....

Inspector .....

SANITARY-INSPECTION RECORD CARD, DALLAS, TEX.
The original is 6x4 in.

and destroy all foodstuffs unfit for human consumption.

The inspection of premises, the elimination of dry closets within the sewered area and the enforcement of the sanitary code form a large part of the work of the inspectors of the bureau of sanitation. A sanitary-inspection card used by Mr. Saville is reproduced herewith. Another phase of the work is the examination and abolition of all places that breed malaria-bearing mosquitoes. Then, of course, there is the usual routine work of handling miscellaneous complaints. A system has been worked out for attending to these complaints as expeditiously as possible by the use of special forms that are referred directly to police, legal or welfare departments for action.

With the coöperation of two or three of the local newspapers it has been possible to give considerable publicity to the work the bureau of sanitation is attempting to accomplish. The efforts of the bureau have also been much assisted by a local society for municipal research, which has a committee on sanitation and housing. This society is composed of department heads and other employees of the municipal government of Dallas, and its aim is "to study and initiate progressive, efficient and economical methods in municipal business affairs and to bring about a closer coöperation of the various departments of the city government." Altogether, the work accomplished in the first year of its existence by the new Dallas department of public health is a fine example of what technically trained sanitary engineers may accomplish in public-health work.

Field and Office

Methods Used in Comprehensive Survey of Buffalo, N. Y.

Until 1903 there were no accurate survey records of the City of Buffalo, N. Y., for use of the assessors—a condition of affairs common to a great many American cities. Such maps as existed were compilations of lot and subdivision surveys, and were as full of inconsistencies as such compilations always are. In 1903 a City Survey Division of the Bureau of Engineering was established, and ever since then a thorough and systematic survey of the city has been under way.

It was intended to base this survey on a triangulation net established by precise methods, but owing to the necessary slowness of this work and the great demands made for special surveys the triangulation has been only partly completed. In order to go ahead with the detail survey without the governing triangulation the surveys are based on closed traverses, about ½ or ¾ mi. square, run through the principal streets. The limiting error of closure for these traverses is 1 in 10,000.

Ordinary methods for city survey work were employed, except that linear measurements were made with a 100-ft. invar tape laid on the ground. Three tapemen were required, one to carry the tape in the middle, for it must at no time be allowed to drag on the ground because of its liability to breakage and injury. Marks cut in sidewalks were used as stations. One monument was set at each street corner—a plain stone shaft, to be ultimately replaced with the type of monument shown in Fig. 1. These future monuments are to be set at street-line intersections, on range with one of the street lines and on an offset of 2 ft. from the other, at intervals of 1,000 ft. The intermediate street lines are to be marked with plain stone monuments set flush with the surface of the ground.

To keep record of the progress of the work, a wall map of the whole city is used, upon which is noted within the lines of each street the number of the field-book and page-number which contains the survey notes of that particular part of the street; also the number of the rolled map upon which the survey notes are plotted in detail.

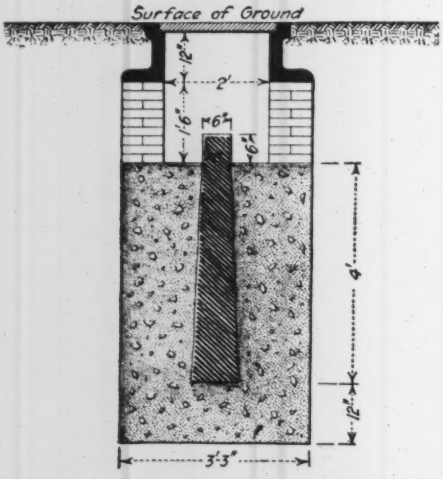


FIG. 1. TYPE OF CITY SURVEY MONUMENT TO BE USED IN BUFFALO, N. Y.

The original record map is drawn on cloth-backed paper on a scale of 200 ft. to the inch, forming rolls 42 in. wide by 20 ft. long (north and south). Street and property lines with original record and deed distances are shown in black; measured distances and courses are in blue (or green originally) and adjusted distances and courses in red. This map shows the frontage of every lot on the street (in many instances the depths of lots are also shown because it was necessary to fix the back corners of the lot to establish the frontage), and also one front corner of any building on the lot.

The city is divided into 14 tax districts, requiring 804 maps and 57 index sheets. These maps are to be bound

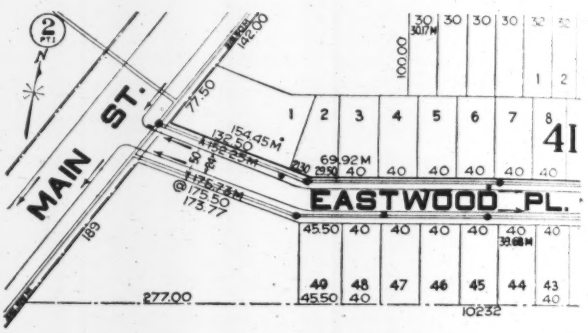


FIG. 2. PORTION OF ASSESSOR'S MAP MADE BY CITY SURVEY DIVISION, BUFFALO

in loose-leaf fashion in 38 parts. The maps are drawn on mounted reinforced Paragon paper sheets 28x36 in. to a scale of 1 in. = 50 ft. On these maps are shown the sublots and their dimensions. Distances from deeds are marked with the letter "D," and measured distances with the letter "M." The original farm-lot numbers are shown in red; the block and sublot numbers are also shown, to assist the assessors in the location of sublots, which are described by block and sublot numbers with particular reference to a certain subdivision map on file in the County Clerk's office.

All existing sewers, manholes, catchbasins and street lights (electric and gas), all of which are paid for in the City of Buffalo by local assessment, are shown on these maps; arrows indicating the direction of the street surface drainage are also shown. Fig. 2 is an example of a portion of such a map. Numbers inclosed in circles at the extreme right and left of the sheet as well as the top and bottom indicate the numbers of the adjoining sheets so as to obviate the necessity of referring to the index page.

As to the disposition of surpluses and shortages in city blocks, it has been found impracticable to apply any set rule that would fit all conditions. In the readjustment

of property and street lines the work is governed to a great extent by existing local conditions. In open country it is comparatively a simple matter to apportion the surplus or shortage over the entire block without causing any material inconveniences.

In many instances lands have been subdivided and maps placed on file in the county clerk's office which indicate that the lots are to be described as being a definite distance from a fixed point of beginning, each lot having a definite frontage except the last lot, which is of a frontage more or less, the evident intention being that the last lot shall bear the surplus or shortage, whichever it may be. In thickly settled portions of the city the property lines are too firmly fixed by reason of long occupation to admit of any radical changes.

In the course of the resurvey of the city, in many localities conditions have been met in which a lot will have an excessive frontage at the expense of another lot some distance beyond in the same block. To attempt a readjustment of the property lines so as to place the excess where it rightfully belongs would cause every intervening property holder to encroach on his neighbor's land, thus giving opportunity for endless litigation far in excess of the value of the land involved. In such cases it is believed preferable to accept conditions as they are.

Wherever it is practicable, an attempt is made so to adjust the property lines as to give every property owner the frontage his deed calls for, even though the readjusted location may disagree with the deed description so far as it refers to the distance from the point of beginning. The Board of Assessors has a record of every piece of property in the city, giving the owner's name, the dimensions of the lot, its location, and land valuation, as well as valuation of improvements. The assessors prefer to use the deed frontages unless objection is made by the owner that the deed frontage is incorrect, when after verification by the Department of Public Works the corrected frontage is used and the changes are made on the map and book of records.

Besides conducting the city survey above outlined, the City Survey Division is constantly called upon to make special surveys for various city purposes, all the survey work of the city being concentrated in this bureau. Fig. 3 reproduces part of the map of such a survey, and incidentally shows a condition of lot layout—incipient city planning—of a particularly flagrant variety. The survey is for a new street to take care in some measure of the haphazard layout of the original sections and blocks of adjoining real-estate developments. The plan at the left shows about half the map in simplified form, while a detail is reproduced in full at the right. The data gleaned in these special surveys are of course made use of for the plans of the systematic city survey.

Buffalo contains about 40 sq.mi., and half of this area has been surveyed. The City Survey Division is in charge of Louis H. Rathmann, Assistant Engineer of the Bureau of Engineering, Department of Public Works, of which George H. Norton is Chief Engineer. The division comprises an assistant engineer in charge of the field party, a transitman, three rodmen (or chainmen) and two draftsmen.

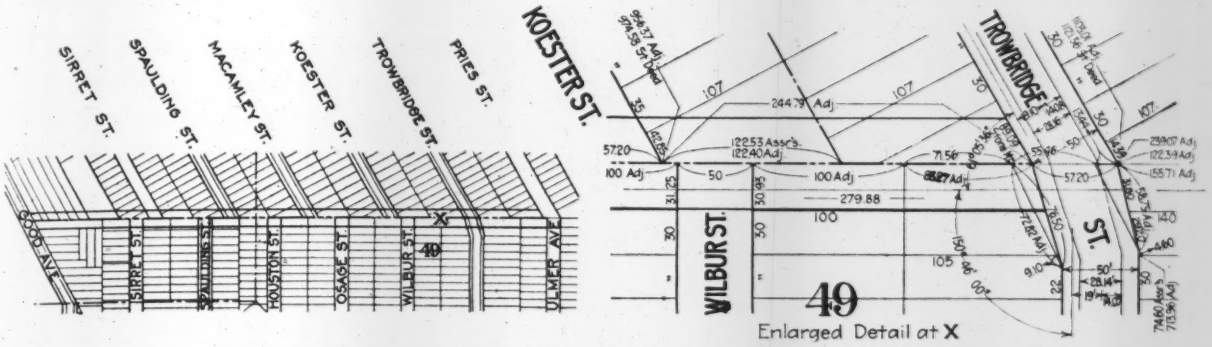


FIG. 3. EXAMPLE OF A SPECIAL SURVEY OF THE CITY SURVEY DIVISION, BUFFALO, N. Y.

Reservoir-Capacity Table

By Arthur T. Clark*

In locating water-works distribution reservoirs on sloping ground it is often convenient to know in advance the dimensions of reservoirs of different depths that will give a required capacity. The accompanying table has been prepared by the writer to show the capacities of square reservoirs of different depths with different side lengths and with different interior slopes. L is length along water edge.

CAPACITIES OF SQUARE RESERVOIRS (MILLIONS OF GALLONS)

L	D = 10 ft.	D = 20 ft.	D = 30 ft.	D = 40 ft.
Interior Side Slope, 1 on 3 (cot θ = 3.0)				
100	0.4			
200	3.0			
300	5.5	8.8	10.5	11.1
400	11.0	17.5	22.5	25.5
500	16.5	29.1	38.3	44.6
600	25.0	44.0	58.5	70.0
700	33.6	61.4	84.1	102.1
Interior Side Slope 1 on 2½ (cot θ = 2.5)				
100	0.4			
200	3.0	2.81		
300	5.6	9.5	11.8	13.0
400	11.2	18.8	24.5	28.0
500	16.9	30.4	40.9	48.5
600	25.0	45.8	62.2	75.9
700	34.1	63.3	88.1	108.7
Interior Side Slope 1 on 2 (cot θ = 2.0)				
100	0.5	0.6		
200	3.2	5.4		
300	5.9	10.2	13.2	15.1
400	11.6	20.0	26.4	30.5
500	17.2	31.7	43.7	53.5
600	25.9	45.9	62.1	76.0
700	34.6	65.2	92.2	115.6
Interior Side Slope 1 on 1½ (cot θ = 1.5)				
100	0.5	0.8	0.8	
200	3.3	6.1	7.7	
300	6.1	11.4	14.7	17.6
400	11.8	21.2	28.4	33.0
500	17.6	33.1	46.6	58.3
600	26.0	49.2	70.4	87.8
700	35.1	67.2	96.4	122.9

The table is also readily applied to circular reservoirs; if L is diameter, capacity is 0.7854 given value. The

*Assistant Engineer to Robert E. Horton, 57 North Pine Ave., Albany, N. Y.

table can be applied to rectangular reservoirs that are not square by adding a water prism $L' - L$ wide, D high and of length $L - D \cot \theta$, where L' is the long side (water edge) and θ is the angle of bank slope.

If it is desired to obtain the dimensions of reservoirs of a given capacity more closely than they can be taken from the table or read by interpolation, curves can readily be constructed by plotting the data given in the table.

Paving Records; Topeka, Kan.

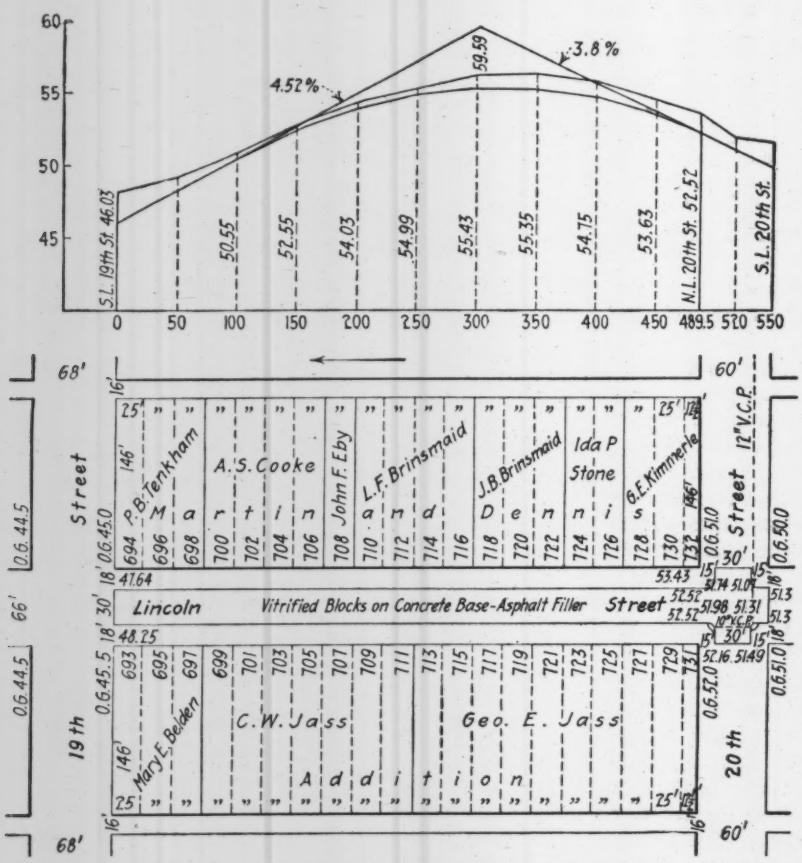
The city engineer of Topeka, Kan., A. R. Young, has adopted the following methods of making and keeping records of pavement construction: A plot of the block or paving district and a profile of the street are made on a single 18x22-in. sheet, of which the accompanying illustration is a small-scale reproduction. These plots are on tracing cloth, divided half and half into plain and standard Plate A cross-section tracing cloth. The scale used ordinarily is 50 ft. 1 in.

These plots serve the double purpose of appraisers' maps and office records. They show all lots and pieces of land in the block or paving district. The file blueprints (described later) also contain in red ink the assessed valuations of the property. Assessments are based on the value of property, not on pro rata frontage alone.

The plots are made by a draftsman, and then boundaries and dimensions are checked twice—once by the office engineer and then by the city engineer. The profiles are plotted from carbon copies of field notes as the fieldwork progresses. The grade lines are put on the profile by the assistant engineer in charge of field work. They are checked and approved by the city engineer and eventually by the Board of Commissioners (Topeka has commission-plan government). Cross-sections, paving areas, curb lengths and other quantities are checked by the office engineer as well as by the field party. All computations are made in bound books, and the record shows the date and initials of the computer and the checker.

As soon as the quantities for each block or paving district are computed, the work as a whole is totaled, the proposal blanks filled out and the contract advertised and let. After the contract is awarded, the detailed estimates of the assessments for each district separately are made, based on the contract prices. Standard forms are provided for these estimates. The computations are made by the office engineer, checked by the assistant engineer in charge of fieldwork and finally checked by the city engineer, who certifies them and transmits the sheets to the city clerk.

By the time this work is done, in ten days or two weeks, the contractors are ready to begin construction. The program starts promptly near the beginning of the new year, and the contractors are at work as soon as weather conditions permit in the spring. There is little trouble in Topeka from contracts delayed until the approach of winter. Blueprints are made of all the plans of the year's work and are bound into books. Near the corner of each tracing has been added a tabulation giving the name of the contractor, date of starting work and the date of completing work, and also a list of the quantities estimated and measured (final) and the unit contract price, all work being let on a unit-price basis. In addition to the plots the file book contains a quantity summary sheet and summary sheet of bids and estimates.



SHEET FROM BOOK OF PAVEMENT RECORDS, CITY ENGINEER'S OFFICE, TOPEKA, KAN.
Actual size, 18x22 in.

The quantity summary gives a list of all the pavements laid during the year, under the proper heading as to the type of pavement, and the final estimate quantities of excavation (cu.yd.), paving (sq.yd.), curb (lin.ft.), paving relaid (sq.ft.), curb reset (lin.ft.), catchbasins, sewers (lin.ft.), concrete walk (sq.ft.), new brick walk (sq.ft.), old brick walk relaid (sq.ft.), manholes, inlets, etc., in each district or block.

The summary sheet of estimates and bids is an extremely valuable part of the file. It contains under the proper heading as to type of pavement a list of all the items bid on during the year. (It is the custom in Topeka to let all the paving of a chosen type or kind in one contract, with the provision that the work is not to be sublet.) The city engineer's

estimate of unit prices and totals is given in parallel columns with each bid received, divided in the same way into unit price and total. The name of the contractor is given in each instance, with an asterisk for denoting the successful bidder. The value of this tabulation, not only as a part of the record, but for making future estimates, can be easily appreciated.

Some Costs of Applying Mortar With the Cement Gun*

The upstream face of the Elephant Butte dam of the United States Reclamation Service is being waterproofed with portland-cement mortar mixed in the proportion of one part cement to two parts sand and applied with a cement gun in a coating about 1 in. thick. Advantage has been taken of rising water in the reservoir to work from rafts specially constructed for the purpose. There are two of these, each 9x13 ft., made of planking on a framework laid on and attached to 16 oil barrels. One raft contains the machine, operator and helpers, small mixing box and a few sacks of cement and sand, while the other is loaded with cement and sand.

The coating is applied in horizontal strips about 10 ft. high and the length of the dam at water level. The surface is first cleaned thoroughly with scrapers and wire brushes and is then gone over with a sand-blast, using coarse sand, passed through the machine to obtain the necessary pressure. This roughens the surface sufficiently to cause the mortar to adhere to it. The surface is then thoroughly moistened with a hose and the mortar immediately applied. The mortar is put on in four layers, each about ¼ in. in thickness. It was found that a thicker coat than this applied on a vertical wall, without reinforcement, would, on account of its weight, slough off before setting. Each layer follows the preceding one before it has attained its final set, and so far there is no indication of a parting of the layers. Numerous samples taken from the face showed perfect adhesion to the concrete, it being impossible in every case to break the mortar from the concrete at the line of contact. The cost per square foot for the first 100,000 sq.ft. of this coating is as follows:

Operating and repair work, including cost of air and water	$0.015
Staging, cleaning wall, moving, etc.	.006
Cement	.024
Sand, including labor, screening and hauling (sand only 20c. per cu.yd.)	.007
Depreciation of gun and equipment	.020
Subtotal	$0.072
Overhead	.007
Total cost per square foot	$0.079

Some experiments were also conducted at Elephant Butte to determine the feasibility and probable cost of troweling the mortar placed by cement gun, with the idea of using the machine for the lining of canals or for repairing lining that had disintegrated and required a smooth finish. The area treated was 75 sq.ft. This was covered with mortar averaging ¾ in. in thickness in 15 min. working time, or at the rate of 300 sq.ft. per hr. for this average thickness. One good finisher troweled 56 sq.ft. in 25 min., or at the rate of 135 sq.ft. per hr. Possibly two men could keep up with the gun. The finishing was not done to a screeded surface. The surface is wavy, but much smoother than a formed surface would be. It could be screeded to a plane, if thought necessary. Care must be taken that too much material is not deposited in one coat, or horizontal cracks will occur, due to settlement. Some cracks appeared in this experiment, and in each one the material was found to be from 1 in. to 1½ in. thick. The cost of this particular experiment, not counting cost of setting up, equipment, etc., was as follows:

Labor	$0.30
Sand, approximately 9 cu.ft.	.90
Cement, 4 sacks	2.00
Total for 75 sq.ft.	$3.20
Per square foot	$0.04¼

Depreciation, staging, moving, etc. would depend on the job, but it is probable that the total cost of placing this coating, including the troweling, under average conditions would not exceed 5c. per sq.ft.

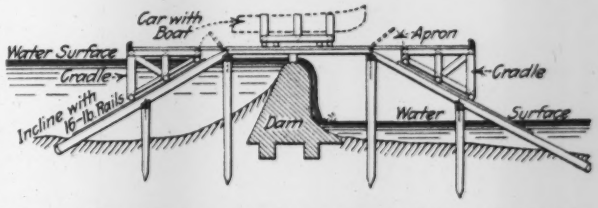
In using the cement gun the sand must be clean, sharp and not too fine. It must not be bone-dry, or trouble with feeding will occur. The air pressure should be about 30 lb. in the gun and the water pressure over 60 lb. It has been figured at Elephant Butte that 30 ft. of free air at 100 lb. pressure at the point of delivery to the gun and 10 gal. of water are required per minute.

*From the "Reclamation Record," March, 1916.

Marine Railways for Hauling Boats Over Dams

Improving the Fox River in Illinois by means of dams is proposed by the Illinois Rivers and Lakes Commission in order to maintain a slack-water level in stretches that are now extremely shallow in dry-weather periods. There is considerable use of the river for pleasure boating, and in order to get boats from one level to the other it is planned to equip each dam with a simple type of marine railway.

The general construction is explained by the drawing. On each side of the dam is an incline with track for a



MARINE RAILWAY FOR PASSING BOATS OVER DAM ON THE FOX RIVER, ILLINOIS

cradle, having a level platform and a track for a car. The cradle, with car upon it, is lowered beneath the water and the boat floated into position on the car. The cradle is then raised and the car rolled across to the cradle on the opposite side, which is lowered into the water. Hinged aprons connect the cradles with the deck of the fixed trestle.

The marine railway would rarely cross directly over the dam in the manner shown. In most cases the incline on the upstream side will be upon the bank, near the end of the dam. Thence the car will run on a narrow-gage track to a suitable point below the dam, where the second incline can be placed. Where there is a headrace, this may provide a location for the inclines. The cost for these marine railways is estimated

at $1,000 each, or $11,000, with $53,960 for dredging at the foot of the downstream inclines. Thus the total cost would be $64,960. The cables of the cradles would be operated by drums driven by electric motors, as electric power is available at most of the dams. Hand operation could be employed where necessary.

A marine railway of this type, but large enough to handle a canal boat 100x17 ft., is being built by the Spring Lake Drainage District, in Illinois. This was ordered by the courts, as the District's levee closed a channel connecting Spring Lake with the Illinois River. The boats will be taken up one incline on a cradle, then turned 180° on a turntable and lowered on the other side.

Town Improvement Records Kept in Card Index

A card-index record of public improvements done by special assessment has been devised by Morgan H. Brightman, city engineer of Elgin, Ill., and was described in a paper presented at the recent annual meeting of the Illinois Society of Engineers. In view of the continual demands for information as to work of this kind the record was prepared to eliminate as far as possible the necessity of frequent reference to original records and also to condense the information to convenient form.

The principal facts and figures concerning the different improvements are recorded on the two sides of standard 4x6-in. filing cards, as shown by the accompanying illustration. One card is used for each improvement. The entire system, alphabetically arranged and filed in a small metal case, occupies an inconsiderable space in the office desk.

In the first four lines on the face of each card a considerable amount of information regarding the improvement itself may be given. For a brick pavement, after the name of the improvement would appear the street in which the work was laid, the points to and from which it extended, the length and width of roadway, nature and thickness of base, make of brick and kind of filler, together with the yardage of pavement and the lineal feet of curb or curb and gutter, and any other quantities of major importance.

CITY OF ELGIN

Special Assessment Improvement Record Card

Morgan H. Brightman, City Engineer

Name			
Extent			
Kind			
Quantity			
Contractor	Address		
Engineer	Assessor		
Est'd cost of construction $	Public Benefit $	Per Cent	
Est'd cost of assessment $	Property Benefit $	Rate	
Total Estimated cost $	Total Roll $	Installments	
Contractor's bid $	Reduction $	Per Cent	
Extras $	Reduced Roll $	Reserved	
Total cost of construction $	Contractor's Bond $	Guarantee	
Contract Record File	Plans	Preliminary Notes	
Docket Ordinance	Profiles	Construction Notes	

CARD FOR RECORD OF SPECIAL-ASSESSMENT IMPROVEMENTS AT ELGIN, ILL.
Card 4x6 in.

On the reverse of the card the column headed "References" is used to good advantage for noting initials and numbers referring to minutes of board meetings, council proceedings, etc., that may be connected in some way to the date opposite. For instance, opposite the date of the adoption of the first resolution the note "B. L. I. III. 125" means that on page 125, Vol. III of the proceedings of the Board of Local Improvements is recorded the adoption of such resolution. The note "D. N. 1/16-1/26" opposite the date of the advertisement for bids would indicate that such advertisement was officially made in the *Daily News* from Jan. 16 to Jan. 26. In this way each card furnishes a record of the bulk of the general data relative to the improvement which it represents. Occasional reference must still be made to more detailed and complete records for information that could not possibly be noted on the cards.

Date of	Month	Day	Year	Reference
1st Resolution				
Notices mailed				
Public hearing				
2nd resolution				
Ordinance passed				
Petition filed				
Roll filed				
Notices mailed, etc.				
Hearing on roll				
Roll confirmed				
Advertisement for bids				
Bids received				
Contract awarded				
Work commenced				
1st voucher				
Work completed				
Approved by Board				
Certificate of costs, etc.				
Notices posted, etc.				
Accepted by Court				
Final voucher				
Roll reduced				

REMARKS:

REVERSE OF CARD

To facilitate this further reference work, blanks are provided on the face of the card, at the bottom, to be filled in with court docket number and the file or index numbers referring to other records in the engineer's office, including field notes, plans, profiles, specifications, ordinances, contract records and copies of all legal or other papers and documents connected with the improvement. An index letter and number are placed at the upper corners of each card to aid in filing and for cross-referencing.

Experiments on the Effect of Hydrated Lime on Concrete

The Committee on Mixing and Placing of Concrete of the National Conference on Concrete Road Building, which met in Chicago, Feb. 15-18, reported some tests that had been made at Milwaukee in the investigation of the proper methods of mixing and placing concrete for roads. Among those tests are a few that throw some light on the question of the value of hydrated lime in concrete. A portion of the report, by Ernest Ashton, of the Lehigh Portland Cement Co., Allentown, Penn., relates to the hydrated-lime investigation. In his report Mr. Ashton says:

> A small amount of work was done to ascertain whether certain statements made relative to the use of hydrated lime were correct, among which the following questions were most pertinent:
>
> 1. Does the addition of hydrated lime increase the mobility of concrete?
>
> 2. With a given angle of chuting and given percentage of water, will concrete flow more readily if hydrated lime is added?
>
> 3. Does the addition of hydrated lime aid in preventing segregation?
>
> 4. Acting as a void filler, will the addition of hydrated lime tend to make the mass more impervious, thereby reducing to the minimum alternate expansion and contraction due to moisture?

A résumé of the results of the tests is given in the accompanying table. The concrete was mixed in a machine mixer and deposited in the molds after being mixed 1 min. and after flowing through a short concrete chute on an angle of 20°. The same methods of manufacture and storage were used on all the specimens. The column headed "Consistency" in the table is somewhat ambiguous, but is the best application that can be deduced from the report.

Further remarks regarding the specimens are that in specimen H-5 there was no separation of materials and that all specimens were poorly compacted; specimen H-10 was more sticky than H-5, and in two of the three tests on which the average was taken was fairly well packed, the other being poorly packed. Specimen H-15 was still more sticky than specimen H-10, and all materials held well together in a mass. All the specimens, however, were poorly compacted. In specimen H-145 the concrete was very soft and easy flowing. In specimen H-1410 the concrete was not so soft as in H-145, but the flow was slower. Of specimen H-1415 the appearance and the flow were the same as of H-1410. Mr. Ashton's conclusions regarding the tests are as follows:

1. No greater freedom of movement of the mass was noticed. In point of fact, as the lime was increased, more difficulty was experienced in getting material into the test forms.
2. The work done shows that, with the same angle and the same percentage of water, concrete did not flow more readily with the addition of hydrated lime; and as the percentage of hydrated lime was increased, the mass became more sticky and did not flow as readily.
3. By far the most important observation made was that the addition of hydrated lime did prevent segregation, so much so as to make the biggest gain that can be derived from the use of hydrate of lime.

Mr. Ashton adds further that the number of specimens made was too small to allow an adequate interpretation of actual values. It is the intention of the committee to carry on during this year a more extensive investigation as to the value of adding hydrated lime. In this connection attention is called to the fact that the Bureau of Standards in Pittsburgh has already started some work along this line.

TESTS OF 6x12-IN. CONCRETE CYLINDERS WITH VARYING AMOUNTS OF HYDRATED LIME AND OF WATER*

No.	% Lime	Lb. of Water per Cu.Ft. of Material	Consistency	Compression,† Lb. per Sq.In. 1 Mo.	2 Mo.
H-5	5	5.3	Sticky	4,090	4,580
H-145	5	6.5	Very easy flowing	2,880	3,250
H-10	10	5.3	Quite sticky	4,630	5,330
H-1410	10	6.5	Quite easy flowing	4,290	5,020
H-15	15	5.3	Very sticky	4,420	5,150
H-1415	15	6.5	Quite easy flowing	4,210	4,620

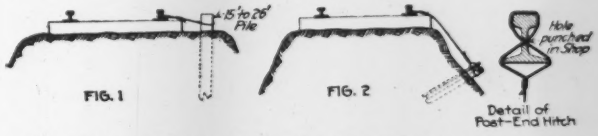
*Tests made on 1 : 2 : 3 gravel concrete. Hydrated lime measured as an added percentage to the weight of cement. Constant slope of chute, 20°; time of mixing, 1 min. †Each figure is the average of three specimens.

Railway-Track Anchors

How to keep a railway track on a fill subject to overflow is one of the problems railway maintenance-of-way engineers sometimes have to face. Two methods have been successfully used on the Missouri, Kansas & Texas Ry. in the Southwest.

The first is illustrated by Fig. 1. Piles are driven 15 to 26 ft. through one shoulder of the fill at intervals of 100 ft. The track is anchored to these piles by galvanized-iron wire looped under and around the tie nearest the pile, being held by the inside flange of the rail. Track anchors of this description saved over two miles of overflowed track in the Brazos Valley, Texas, in the spring floods of 1915. Since then track anchors have been used in many localities on this line. The type described is objectionable for several reasons: It is costly, the anchors are too far apart, and the wire buried in the ground at the ends of the ties rapidly rusts out.

Another form of track anchor which is giving satisfaction is shown in Fig. 2. This consists of pieces of old rail 5 or 6 ft. long driven perpendicular to the slope of the fill. The sections of rail are cut and punched at the shops and are placed by section gangs. The hole is first started with a post-hole auger, and then the rail is driven home with a similar section of rail swung, or raised and dropped, by two men—one at each end. These rail posts are driven every two rail lengths, or every 66 ft. There is a hole punched near the top of the post, and through this a ⅜-in. galvanized wire guy line (or wire rope) is passed, as shown in the little detail, Fig. 2. This line passes over the shoulder of the fill and is fastened to the track, as in Fig. 1.



RAILWAY TRACK ANCHORS, MISSOURI, KANSAS & TEXAS RY.

The foregoing data were obtained from A. A. Matthews, Engineer of Maintenance-of-Way, Missouri, Kansas & Texas Ry., Dallas, Tex.

Cost of Cement Testing

By Frank H. Carter*

It recently became the duty of the writer to equip and install a small-cement-testing plant to handle about 5,000 bbl. of cement. The cost of the apparatus, together with a statement as to the cost of the testing, per barrel, forms a basis for an interesting comparison of the cost of the same work by cement-testing laboratories.

COST OF APPARATUS

1 enamel pan	$0.55
1 5x5-in. glass	.10
1 2-qt. kerosene can	.05
1 bottle	.05
1 funnel	.15
1 funnel	.05
1 enamel pan	.55
1 piece glass	.40
8 pieces glass	.32
1 thermometer	.50
1 50 sieve 1 100 sieve 1 200 sieve 1 20 sieve 1 30 sieve	19.00
1 one-section mold	3.20
1 two-section gage mold	6.40
1 three-section gage mold	9.60
1 testing machine	144.00
1 Vicat needle	16.00
1 specific-gravity bottle	2.75
1 keg sand	6.25
	$209.92
Cost per barrel ($209.12 ÷ 5,000)	4.20c.

The time required for the various operations is shown in the appended table:

	Hr.
"Normal consistency"	1
Determining per cent. on 100 sieve	1½
Mixing batch for forms	1½
Breaking 30 briquettes (10 min. for each)	5
Sampling from carload	2
Specific-gravity determination	½
Mixing cement for composite sample	¼
Obtaining water	¼
	12

*Designing and Supervising Engineer, Cliftondale, Mass.

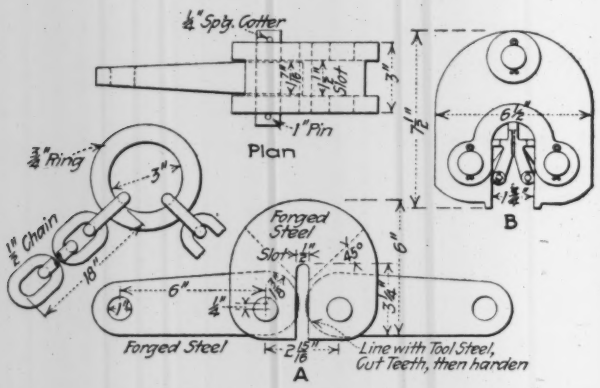
The operator was paid $15 per week of 48 hr., or 31.3c. per hr. As he could test 200 bbl. of cement in 12 hr., there was a wage charge of 1.87c. per bbl. The first cost of the apparatus, per barrel of cement, was 4.20c., which makes the total cost per barrel 6.07c. The apparatus is, however, on hand for use by the city engineer's department.

The operator had no previous experience whatever in cement testing before doing this work and was "broken in" by a recent graduate who had taken a laboratory course in cement testing. It should be stated, in view of the low cost of doing this work, that the operator was employed on the force-account cost data when not engaged in testing cement, and his time while on that work was deducted from the cost of the cement testing.

Clamps for Pulling Steel Piles

In pulling steel sheet piling the use of a hook in a hole in the pile is not always effective, as the hook may tear through the metal without loosening the pile. Two forms of clamps designed to give a sure grip on the pile are shown herewith. They are manufactured by the Vulcan Iron Works, Chicago.

In the design shown at *A* the web of the pile is gripped between the eccentric cams formed by the ends of two



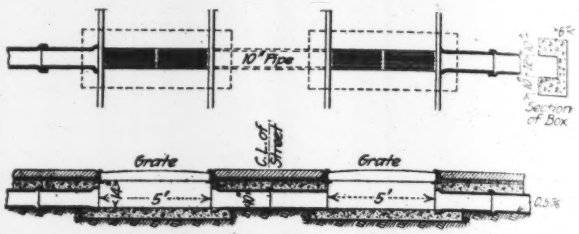
CLAMPS FOR PULLING STEEL PILES

levers journaled in a heavy steel forging. A chain sling is attached to the outer ends of the levers; and as the pulling strain is applied to the sling, the serrated faces of the cams grip the web between them. In the second design *B* the arrangement is more compact. The two jaws, or grips, are carried by arms pivoted in the horseshoe frame, so that the downward pull on the jaws (due to the resistance of the pile) tends to draw the arms inward, thus forcing the jaws together.

NOTES

Roof Trusses Spaced Close to Carry Shafting—In a new shop building just being erected for the Fox Machine Co. at Jackson, Mich., which has three 30-ft. bays, the roof over the side bays is carried by trusses spaced 8 ft. on centers. The columns, both wall and interior, are spaced 24 ft., and longitudinal trusses extend between them to carry the side-bay trusses. The close spacing of the trusses was adopted with a view to providing for the support of shafting and of pipes for heating and ventilating. The bottom chords are made of two channels spaced 1 in. apart back to back. The pipes rest upon them, and the shafting hangers are attached below. No intermediate supports are required. The managers believe that changes in shafting arrangement, as required when shifting a machine to another location, will be greatly facilitated by this fact.—A. Brett.

Culvert Under Street with Car Tracks—A surface-water drain for streets having no storm sewers sometimes has to cross a street with car tracks. It is common practice in a number of Western cities that have no storm-water sewers to build surface-water drains as deep gutters and to carry

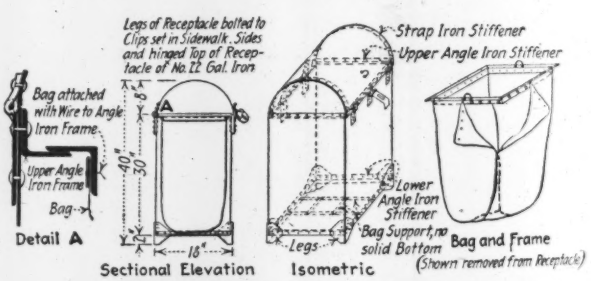


CULVERT FOR CARRYING STORM WATER ACROSS STREET WITH CAR TRACKS

them across street intersections by a dip or gully, through the crown of the cross-street. This plan is obviously bad from the point of view of traffic and of course impossible where the cross-street has car tracks. Fort Worth, Tex., has no regular storm-sewer system. What surface water does not run in the gutters is conducted into drains laid under the gutters or under the sidewalk parking. From these it either runs out into the gutters again at low points or is conducted into a few specially constructed storm sewers. To get across streets with car tracks, the construction shown in the accompanying sketch is used. The cast-iron grating rests on the rail flanges and is loose enough to be conveniently removed for cleaning the concrete culvert. The same type of grating and inlet is used where there is a sump or low spot in the railway-track area to be drained. There is an apparent benefit in storm-water drains laid under the gutters on both sides of the street—that is, the facility afforded of draining the pavement foundation. Fort Worth has some excellent wood-block pavement laid on a 1½-in. sand cushion and with sand-filled joints, which have never given any trouble from expansion or bulging. The street thus paved has a storm-water drain under each gutter, and it is not unlikely that this has served to keep both the subsoil and the sand cushion perfectly drained. F. I. Von Zuben is City Engineer of Fort Worth.

The Engineering Library of the Civil Engineers Society of St. Paul has been given to the St. Paul Public Library to be used as the nucleus of a notable technical reference library. This donation consists of 1,100 volumes, including many valuable reports and periodical files.

Street-Refuse Receptacle—The accompanying sketch illustrates a new type of street-refuse receptacle designed by the office of the Director of Sanitation (Charles Saville), Dallas,


Legs of Receptacle bolted to Clips set in Sidewalk. Sides and hinged Top of Receptacle of No. 12 Gal. Iron
Strap Iron Stiffener
Upper Angle Iron Stiffener
Bag attached with Wire to Angle Iron Frame
Upper Angle Iron Frame
Bag
Detail A
Sectional Elevation
Lower Angle Iron Stiffener
Legs
Bag Support, no solid Bottom
Isometric
Bag and Frame
(Shown removed from Receptacle)

STREET-REFUSE RECEPTACLE DESIGNED BY THE BUREAU OF SANITATION, DALLAS, TEX.

Tex. It is made of 22-gage galvanized iron, properly painted and labeled. It is reinforced top and bottom with angle-iron stiffeners. The cover is a semicylindrical hood open at the ends. The receptacle is placed parallel to the sidewalk, with the supposition that people will cast refuse in at the ends as they pass. The receiver is a detachable bag, which with its frame may be removed and emptied. The price of a sample receptacle was $7, but in buying in quantity the price would be much less.

Editorials

How Civilian Engineers May Study Military Practice

Since a course of lectures on military science was enthusiastically entered upon by over two thousand engineers in New York City, as noted in these columns, we have received some requests from engineers in other localities to reprint the New York lectures or to publish a series of articles by some recognized military authority, covering the same ground. The reasons why we have not thought it best to do this is that the matter presented in these lectures and in any such series of articles, if it is really to educate engineers for practical military work, must be extremely elementary, from the military standpoint, and must therefore be based directly or indirectly on published textbooks readily available to anyone.

The same ground that such lectures attempt to cover can be covered even more completely by a brief reading course. Indeed, such a course has now been recommended for those attending the New York lectures, and we understand that the committee in charge of these lectures will probably advise those attending the lectures to follow such a course instead of reprinting the lectures for them.

Instead of taking space in these pages for such elementary textbook matter, therefore, which would necessarily have to be given in much abbreviated form, we can render our readers greater service by recommending a list of books suitable for home study and equally available for the engineer out of reach of any center where military lectures may be attended or for the engineer who wishes to supplement his attendance on lectures by the study of textbooks. The following books are selections from a list drawn up by the Chief of Engineers of the United States Army, with two other recommended works. The full list will be printed in the next engineering-literature section of *Engineering News*, Mar. 16, 1916.

On Military Policy and the Conduct of War—"Field Service Regulations, 1914"—United States Army (may be obtained from the Superintendent of Documents, Government Printing Office, Washington, D. C.; 60c.). "Elements of Strategy"—Fieberger. United States Military Academy (obtained from the book department of Army Service Schools; 75c.).

On the Organization and Duties of Engineer Troops—"Studies in Minor Tactics, 1915"—Army Service Schools (may be obtained from the book department of the Army Service Schools, Fort Leavenworth, Kan.; 50c.). "Infantry Drill Regulations"—United States Army (may be obtained from the Superintendent of Documents, Government Printing Office; 60c.).

On Field Engineering—"Notes on Field Fortifications"—Army Field Engineer School (may be obtained from the book department of the Army Service Schools, Fort Leavenworth, Kan.; 30c.). "Engineer Field Manual"—Corps of Engineers, United States Army. Third edition, 1909 (may be obtained from the Superintendent of Documents, Government Printing Office, Washington, D. C.; $1). "Military Topography for Mobile Forces"—Sherrill. Second edition (may be obtained from the publisher, Banta Publishing Co., Menasha, Wis., or the book department of the Army Service Schools, Fort Leavenworth, Kan.; $2.25).

On the Coast Artillery—"The Service of Coast Artillery"—Hines and Ward (may be obtained from the publishers, Goodenough & Woglam Co., 122 Nassau St., New York City; $3.50).

Can Rock-Fill Dams Stand Up Under Overflow

The conditions necessary in the construction of a rock-fill dam, in order to make the structure safe, were discussed editorially in the issue of Feb. 10, in connection with the recent disastrous failure of the Lower Otay dam near San Diego. A later detailed report of the Otay dam failure, by Roy A. Silent, was printed in the issue of Feb. 17, and elsewhere in the current issue is published a series of papers presenting the views of four engineers on the design of rock-fill dams.

The disastrous failure at Otay should not discredit the rock-fill dam as a type. There are unquestionably many locations where a rock-fill dam can be built more cheaply than any other type. There are places where the cost of the cement necessary to build a masonry or concrete structure would be prohibitive. There are places where the rock available at the site or within reasonable distance would be unsuitable for use in a masonry structure and yet would make a substantial rock-fill. The rock-fill again has notable advantages in a country subject to earthquake shocks. It is important, therefore, for any engineer interested in dam construction to know exactly what happened in the Lower Otay dam failure and what lessons in the design of such structures are to be drawn from that disastrous occurrence.

The account of the dam's failure, by Mr. Silent, confirms with somewhat greater detail the earlier story by John W. Bacon in the issue of Feb. 3. It deserves thoughtful study in connection with the careful analysis of the stresses and the conditions in the interior of a rock-fill dam subject to overflow, given in the paper by Ralph Bennett elsewhere in this issue.

According to Mr. Silent's report, the rising water of the reservoir reached the crest of the dam at 4:45 p.m. At that time water was escaping from the reservoir through the spillway at a rate of 2,400 cu.ft. per sec. and also through a 48-in. outlet pipe under a head of about 80 ft. What the rate of rise of the water in the reservoir was at that time is not known; but it cannot have been rapid, for the reservoir covered an area of 860 acres. Other figures in Mr. Silent's paper indicate that the reservoir level rose at the average rate of about 7 in. per hr. on the day of the failure. The sequence of failure was as follows: The water reached the top of the dam at 4:45 p.m. This water apparently percolated down through the rock-fill and five minutes later was issuing in small streams from the lower face near the bottom and starting the caving of this lower face. This rock-fill then "quickly melted away," leaving the core wall unsupported, so that the water pressure on the upper side tore it in two at 5:05 p.m.

It appears from these figures that the destruction of the rock-fill must have taken place with a depth of not over 2 in. of water flowing over the crest. Such a "melting away" of a huge pile of rock can only be accounted

for by taking into consideration the pressure to which it was subjected as a structure, as well as the water flowing through it. The mere falling by gravity of a thin sheet of water, not over a couple of inches in thickness, through this loose rock-fill, had it been standing by itself, would have had no effect upon it. The engineer who should attempt to excavate a pile of such rock by simply pouring a stream of water on the top would find no results from his efforts. In order to account therefore for the very rapid settlement of the Lower Otay rock-fill under this trifling amount of water the heavy stress to which the whole mass was subjected must be considered—a stress undoubtedly approaching its crushing strength. The water percolating through the mass acted as a lubricant and also by the hydrostatic pressure set up in the lower part of the dam helped to force out the rocks on the lower face of the fill, which were in unstable equilibrium anyway by reason of the steep slope on which they lay.

In the very interesting analysis of various rock-fill dams, given elsewhere by Mr. Sellew, reference is made to the coefficient of stability, or the quotient of the total weight of the rock mass divided by the water pressure against it. This coefficient should not be confused with the coefficient of friction of a concrete or masonry dam, to resist sliding upon its base. So far as sliding is concerned, the resistance of a rock-fill dam with its rough surfaces is doubtless greater than that of a masonry dam, which may slide upon a smooth base, as did the unfortunate dam at Austin, Penn.

The failure of a rock-fill dam, as illustrated by the Lower Otay, appears to be a gradual yielding or settlement of the whole mass of broken rock. Such yielding and settlement have been noted in rock-fill dams to a small extent under pressure alone, without the action of water upon the mass. At the Lower Otay the lower face of the dam was so steep that the rocks lying on it were about ready to slide. The pressure of the full reservoir, when combined with the lubricating and hydrostatic action of even a small amount of water in the rock mass, was sufficient to start this lower face to sliding and caving to such an extent that within ten minutes the rock fill had settled down so far that the core wall projecting above could not stand the pressure against it.

If the results at the Lower Otay were to be the sole guide, it might be concluded that the rock-fill dam is even more susceptible to injury, if its crest is overtopped by a flood, than an earth dam. A well-built structure of clay, with its slopes well sodded, would probably have stood considerably more overflow, before eroding to the point of breakage, than did the rock pile at Lower Otay.

On the other hand, some of the early and enthusiastic advocates of the rock-fill dam went so far as to claim that it was especially adapted to resist overflow. Some of the older readers of *Engineering News* may recall the plans of the Nicaragua Canal Co. to control the flow of the San Juan River by building rock-fill dams in its bed, somewhat after the plan adopted by Mr. Corey many years later in damming the flow of the lower Colorado, when it escaped to the Salton Sea. The opposite point of view is presented by some of the contributors to the symposium elsewhere in this issue, who argue that under no circumstances should a rock-fill dam be expected to stand overflow. They contend very properly that for a rock-fill dam, as for an earth dam, ample spillway capacity should be provided to take care of floods.

There is no doubt whatever that ample spillway capacity was not provided at the Lower Otay dam. The area tributary to the Lower Otay reservoir was 139 sq. The spillway located beside the dam was 38 ft. wide and 8 ft. deep. In contrast with this the Escondido rock-fill dam in the same locality collects water from a drainage area of only 8 sq.mi. and has a spillway 25 ft. wide and 5 ft. deep.

But while it is easy to say in the light of recent events that the spillway at the Lower Otay dam was insufficient, what engineer would have been able to prove to the people of San Diego, prior to the recent flood, that the spillway at the dam was inadequate? Remember that in all the 20 years the dam had been standing, the reservoir had never been filled to within 15 ft. of the spillway floor and not a drop of water, therefore, had ever run through the spillway.

Mr. Jorgensen, in his paper in this issue, says that "the covering of the crest and downstream face of a rock-fill dam to enable it to pass a sheet of water without injury would undoubtedly be more expensive than provision of the same discharging capacity in a spillway." There are situations where this would be true. There are other locations, however—such as dams in narrow cañons—where a wide spillway can only be provided at great expense.

The most notable failure of a rock-fill dam prior to the washing away of the Lower Otay dam was that of the Walnut Grove dam in Arizona. It was overtopped by a high flood in the night, and its failure caused the loss of 129 lives. Like the Lower Otay dam, it was built with a very steep downstream slope, which must inevitably have been rapidly washed away when the water passed over it. Had it and the Lower Otay dam been built with flat slopes of, say, 2 on 1, it is quite likely that both might be standing today, notwithstanding their deficient spillway capacity.

It must be borne in mind that in building any dam of this type the engineer must face the question, What is sufficient spillway capacity? In a large part of the arid regions phenomenally heavy rainfalls occur at long intervals. If an engineer were to provide spillway capacity sufficient to care for these very rare rainfalls, he would be censured as extravagant by those furnishing the money for the work. In such cases, however, it may be possible for an engineer to secure the necessary funds to give his dam a reasonably safe downstream slope and place sufficient surplus rock on it so that the mass will stand considerable settlement under the action of water without leaving the water-tight face unsupported and also to protect the slope by more or less regularly laid stones that will stand some flow over them without displacement.

Advantages of Serial Bonds

There is little doubt that the people of the United States are about to undertake much larger expenditures on road construction than ever before, and probably most of the funds for this work will be obtained through bond issues. A bulletin has recently been sent out by the officers of the American Highway Association urging upon county officials that highway-construction bonds should be issued on the serial plan instead of on the more common sinking-fund plan, and at the recent Pitts-

burgh meeting of the American Road Builders' Association a resolution to the same effect was adopted.

The history of sinking-fund accumulations shows not a few cases where moneys have been diverted from these funds to other uses. There is always a chance, greater or less, of defalcation and loss of securities kept in a sinking fund, and the safer plan by far is to apply directly the money collected every year toward the payment of debt. While experience with sinking funds in the hands of municipal and state officials has been unsatisfactory, still worse results would be likely with county sinking funds, for it is pretty well known that the standards of honesty and efficiency in county government throughout the United States average below those in either municipal or state government.

The argument formerly made in favor of the sinking-fund plan was that bankers and investors preferred to purchase long-term rather than serial bonds. Perhaps this was true years ago, but recent experience indicates that bankers and investors prefer serial bonds, on account of their greater safety. An additional argument for serial bonds is the loss that occurs if money that should be invested in the sinking fund is allowed to lie idle in the banks for a time. According to a recently published statement, the average uninvested amount in New York City sinking funds last year was more than $10,000,000.

A further argument in favor of the serial-bond issue is that the interest payments decrease every year with the diminishing principal. In case of bonds issued for road construction, where the expense of maintenance is great, the total value remaining in the road at the end of the usual bond period is problematical. It is especially desirable that the cost of the road should be paid for by those who benefit by it during the period of use and not be left as a burden for the taxpayers to carry after the roads are worn out or require heavy maintenance expenses.

Reducing the Time for Street-Car Stops in Cities

The old saying that time is money is remarkably illustrated by the statement that the Interborough Rapid Transit Co., of New York City, has expended $150,000 on improvements in operating apparatus and methods in order to reduce by one second the duration of the stops of its express trains.

This sum seems an enormous one to spend for a very short saving in time. When, however, one multiplies the number of express-train stops per annum by the number of passengers carried on the average express train, obtaining in this way the aggregate saving of time to the public, and then compares with this figure the annual interest and sinking-fund charges on the investment necessary to save one second in the average express-train stop, he finds that a very small sum is being expended to save the traveling public an enormous amount of time. The same problem may be figured also from the company's point of view as to the value of the time saved in the increased service obtained from the enormous investment in the railway and in rolling stock, and also on the basis of the increased traffic earnings possible on a line now crowded to the limit of its capacity.

It would be of great public benefit if such methods of computing the value of time saving were adopted by the managers of street railways throughout the country. One can see in almost any city in the country, especially those where pay-as-you-enter cars are in use, street cars standing waiting at the corners where traffic is densest while long lines of men and women file on board and the conductor makes change with each successive one for the payment of his fare. Where, however, bunches of tickets are sold at a reduction, as in Washington and Montreal, for example, four passengers out of five walk past the fare box without stopping, dropping a ticket as they pass. The street railways of other cities by selling tickets at enough reduction from the regular fare to cause their general use in the place of cash fares, would profit in the time saved in the movement of the cars. In addition they would gain by the increased use of the cars which results both from the ticket system and from more rapid car service.

Steel Passenger Cars from a British Viewpoint

Nobody in the United States doubts that the steel passenger car has come to stay. In fact, the building of wooden passenger cars in this country has practically ceased. It is surprising, therefore, to learn that in Great Britain the steel car is still looked upon with grave suspicion as a radical and dangerous innovation. The British Board of Trade has been investigating the recent bad accident at Jarrow, where the flimsy English wooden cars were badly smashed. "The Engineer," London, comments on the accident as follows:

> We may turn now to the subject of metal coaches. The all-metal vehicle is an unsatisfactory ideal. It is difficult to construct, is not easily decorated, and is very susceptible to climatic changes. Steel coaches are understood to resist better the evils of collisions and derailments, but they are not without objections that are not found in wooden stock. One does not relish the idea, for instance, of having to be rescued from an all-steel car by an oxyacetylene blow-pipe cutting the sides of the carriage open. The rendering of wood noninflammable is a better idea, and here, again, the London & North-Western has made a move. It has erected at Wolverton a fireproofing plant at a cost, we believe, of about £15,000. This plant consists of two cylinders and the other necessary equipment and can deal with about 150,000 cu.ft. of timber per annum. The Midland Co. has a similar plant.

The recent disastrous rear collision of the New York, New Haven & Hartford R.R., where a passenger train ran past danger signals and struck the rear of a standing express train, has furnished American engineers a new demonstration of the great value of the steel car in safeguarding human life. The car hit was a steel Pullman, and only six of the passengers in it were killed. Had this car been of wood, there can hardly be a doubt that the heavy locomotive would have telescoped it for its entire length. This experience is only a repetition of the history of all recent American railway accidents where steel cars were involved.

Nevertheless, from a British viewpoint the steel car is an "unsatisfactory ideal," and processes for rendering wood noninflammable are "a better idea"! Such processes were thoroughly exploited in the United States a dozen years ago and have been generally abandoned. Why British conservatism should cling to such fads and oppose steel passenger cars is a puzzle we will not attempt to solve, unless it is because of a tenacious respect for the decorative style peculiar to the wooden car architect. The steel car, it will be noted, "is not easily decorated."

Letters to the Editor

Proposes a New Pile Formula

Sir—Concerning A. M. Knowles' article on suggestions for steam-hammer pile formulas: Practical engineers no longer believe that piles can be driven by mathematics; hence, why bother to work out mathematical formulas? In the days of the ordinary drop hammer the old *Engineering News* formula $p = \frac{2wh}{s+1}$ was used. In recent years when placing important pile foundations, no formulas have been used, but instead, the piles have been tested by actual loading before the structure was built upon them.

However, in less important work the engineer no doubt wishes to convince himself that the piles are being driven to the proper penetration to give the required safe load. In this event a suitable formula applicable to the method used in driving the piles is required by some engineers. The formula given herewith has been found very useful by the writer in predicting the safe load on a pile, where the subsoil is known:

$$p = (Sa + R)l$$

in which

p = Safe load per pile, pounds;
S = Skin surface of pile under soil, square feet;
l = Length of pile in ground, feet.

For loam and silt $a = 1.7$, $R = 47$; for moist clay $a = 7.4$, $R = 96$; for sand or clay $a = 19.2$; $R = 203$; for coarse sand and gravel $a = 29$, $R = 300$.

This formula seems more reasonable than one based principally upon the mechanical operation of the hammer, as in the analysis of the case the hammer is simply the contrivance used to place the pile in position under the ground.

When driving piles, it is assumed that the field engineer will use his judgment to determine, if possible, whether a pile has hit a boulder, whether it is brooming underneath the ground or has split or sheared off where the eyes cannot see. Good judgment in this respect comes only from actual experience. To apply too many mathematical formulas to pile driving is in the nature of a subterfuge. CHARLES F. ROHDE.

Brooklyn, N. Y., Feb. 26, 1916.

Engineers Discuss Complaint of One of the Bidders

Sir—Lo, the poor engineer! He is tried and found guilty of making his estimates too high and of making them too low. He is a rascal in the first case and a criminal in the second. He is found guilty by his contractor friends, who object to making their own estimates, and is sentenced to "a punishment fitting his crime" by so eminent an authority as *Engineering News*—and all that without any representation at court.

I grant you that there are incompetents and even selfishly dishonest men calling themselves engineers, but let us give them a fair trial before we condemn them.

Have you ever, as an engineer, made a reasonable estimate and had a contractor refuse to bid because it was too low? I have. Have you then made other similar estimates and had the same contractor bid so much below you that your estimate looked foolish? I have. Did he then tell you he had to do it to meet threatened competition from another contractor? He told me that.

Did you ever make an unusually high estimate on a certain class of work on account of unusually difficult conditions and insert in your specifications that "bidders must assume responsibility for their bid by personal inspection of the ground" and then have some contractor make a foolishly low bid even for normal conditions? A friend did. Were you then charged in the District Court with making a criminally fraudulent high estimate to protect your contractor friends? My friend was.

Are you limited by law in the amount you can spend on an improvement by the amount of your preliminary estimate? Most of us are. Are the conditions such that you can tell within close limits what the actual costs will be? They generally are, but not always.

Are these not all pertinent questions, and do they not make our work difficult enough without the acceptance of such biased estimates as presented by "One of the Bidders"?

I do not know anything about the contract on which he bid, except as he describes it; but let us analyze his estimate on his own statement of conditions:

First, "cement costs $1.50 per bbl., with the usual 10c. per bag refund"; but it takes four sacks to hold a barrel, so the net cost of cement is $1.10 instead of $1.40. Then with 10c. added for cartage and storage and testing, the cost is $1.20 instead of $1.50, and the cost per cubic yard is $1.80 instead of $2.25.

Second, "gravel and sand were offered at 90c. per cu. yd." If the gravel contained nothing smaller than ½ in., we could not put more than 1¼ yd. of combined material in a yard of 1:2:4 concrete. Gravel grading down to ¼ in. would require even less per yard of concrete.

As to 20% loss, why should there be over 5% in a well-managed organization? The contractor would surely require from the dealer that the sand and gravel be clean enough to pass the specifications, so the loss from condemned material would fall on the dealer. A total allowance of 1.3 cu.yd. of sand and gravel should be fair. This amounts to $1.17 instead of $1.62.

The present quotation of steel at the mill for ½-in. bars is $2.05. Freight is probably in the neighborhood of $2, and the bending, placing and wiring of steel in the class of work described should be done for $8 per ton. This amounts to $2.55 per 100 lb., or $3.32 per cu.yd. of concrete instead of $3.90.

The retaining walls probably contain about 60% of the concrete above the footing, or 16 cu.ft. to the cubic yard of total concrete. Then if 27 sq.ft. of forms is required to the cubic yard, the average thickness of the wall is 16 ÷ 13.5, or about 14 in. Does it really cost "One

of the Bidders" 10c. per square foot to build that class of forms? I have actually worked for a contractor who did it for 6c. I will admit there might be unusual features unknown to me, which make the work more expensive. I am assuming work of average difficulty, so let us figure it at 8c. and find $2.16 per sq.yd. instead of $2.70.

Mixing and placing and water and waterproofing could cost $1 per yd. under difficult conditions. Overhead expense is a fair item as estimated. We have then for our estimate:

1.5 bbl. of cement at $1.20	$1.80
1.3 cu.yd. sand and gravel at 90c.	1.17
130 lb. reinforcing steel at $2.55	3.32
Mixing and placing	1.00
Forms	2.16
Overhead	.50
Total	$9.95
Profit, 15%	1.50
	$11.45
Royalty, 10%	1.28
Total estimate	$12.73

True, this is considerably more than $7.65 per cu.yd., but it also is 14% less than "One of the Bidders" estimated, without changing the profit in any way except to increase the per cent. in that item; and it is fairer to both parties than either $7.65 or $14.92.

We all agree that the engineer's estimate should be fair and allow for a reasonable profit. "One of the Bidders" made his estimate much as an old, experienced contractor told me that he did. He said he was like the old backwoodsman who always first aimed squarely at the squirrel sitting on a limb and then allowed a little for crawl. But if an engineer "allows a little for crawl," it is almost a certainty that some bidder anxious for work, or even more anxious to keep competition shut out of the territory, will underbid his estimate so far that he is placed in the class of those who make their "estimate absurdly high in order to enhance their own reputations."

What is the fair course, then, to pursue? Is it not, as you suggest, that the engineer make a fair estimate; and if he is not familiar enough with all the details to do so, should he not ask that competent consulting advice be furnished him?

But how can the engineer protect his reputation, even by this course, from the erratic bidding of the many who "start in the direction of the well-populated graveyard of their profession" by "filing bids without attempting to make even a rough estimate of the cost," to say nothing of the pernicious combination of bidders to hold prices too high or the absurdly low bid to shut out dangerous competition? AN ENGINEER.

Kansas, Feb. 8, 1916.

Sir—In your issue of Feb. 3 you published a letter under the heading, "Contractors Should Not Rely on Engineers' Estimates," wherein "One of the Bidders" complained of the injustice done contractors by the inaccurate advance estimates of cost prepared by engineers.

While the engineering profession will, I am sure, agree with your editorial comment that the engineer's estimate should be approximately accurate and include a margin of profit for the contractor, the fact remains that there is likely to be nearly as much variation in engineers' estimates as in those of contractors. As the engineer's estimate is not usually prepared for the convenience or guidance of the prospective bidders, the contractor who submits a bid prepared by the "shaving" process deserves what he gets.

I must, however, take exception to the statement that "his estimate, which he offers, seems to be fair" and, without going into details, beg to call your attenion to several discrepancies in the figures given.

The correspondent states, "The cost of standard cement f.o.b. cars was $1.50 per bbl., with the usual 10c. per bag refund." Yet in making up his estimate he used $1.40 as the net price for a barrel of cement, either with the idea that only 25% of the bags would be returned or unintentionally having confused the units employed.

He also says: "The walls had to be waterproofed on both sides . . . Approximately ½ sq.ft. of waterproofing was required per cubic foot of concrete," or 13½ sq.ft. per cu.yd. Assuming this to be the area of both faces of the wall per cubic yard of concrete, it is not made clear why 1 sq.ft. of forms for each cubic foot of concrete is required, or 27 sq.ft. per cu.yd., which is twice the area given above. Without details it is impossible to verify either of these figures; but if the first is correct, the cost for forms would be $1.35 lower than that given.

I make no attempt to justify the engineer's estimate of $7.65 per cu.yd.; but unless my deductions are very much in error, your correspondent's estimate could be cut $12,000 without reducing the item of "Profit"—and probably some more.

Anyone who has studied bid tabulations on miscellaneous jobs during the last few years will have noticed many in which the highest bid was from 50% to 100% above the lowest bid, with the engineer's estimate frequently somewhere in between. I know of one instance where 12 bids were received ranging from $45,391 to $92,496, the engineer's preliminary estimate being $59,674. It may have been a coincidence that one of the bids was $59,654.

Personally, I incline to the belief that the tendency of engineers estimating on public work is to over-estimate rather than under-estimate and that frequently "Profit" is included in every item and then added as a percentage of the whole.

It is probably true, however, that even with the best of intentions, engineers will sometimes fail to include certain elements of cost, the omission of which may mean the difference between profit and loss to the successful bidder. LEICESTER DURHAM.

Municipal Building, New York City, Feb. 14, 1916.

[The preceding two rebuttals of the letter entitled "Contractors Should Not Rely on Engineers' Estimates" were submitted for reply to the writer of the letter. His comment follows.—Editor.]

Sir—In my first letter the cost of cement should read $1.80 per bbl. with the usual 10c. per bag refund, instead of $1.50 as printed. The work was strung out along a line nearly a mile long, partly on the river bank, partly on a dirt street and the rest of the way through a garbage dump.

The location, together with the specifications that the *minutest* particle of foreign matter in either the sand or the gravel would cause its rejection, accounts for the heavy allowance for waste of these items. Some of the walls were 33 ft. high, and of necessity a section must be completed before another one could be started, which required well built and strongly braced forms.

The plans gave only approximate elevations for the base of the wall and no assurance was given that the steps

in the foundation would be in any fixed amount, the height and frequency of the steps would be determined as the excavation proceeded. The wall showed a coping on both face and back, precluding the projection of any forms above the wall. The location of the work is subject to overflow of the river. Only that part of the wall that showed above the ground was to be waterproofed. The work was to be started at once and pushed with vigor, which precluded waiting on mill deliveries of steel.

The courts have held that the engineer is the engineer for the contractor as well as the engineer for the contractee. If this is the case, can we not by the same process of reasoning make the engineer the engineer for the *bidders* as well? "ONE OF THE BIDDERS."

Military Fitness of New York State's Engineers

Sir—Referring to the letter signed "Vet," in your issue of Feb. 24, 1916, in regard to the military training of the engineers in the employ of New York State, I wish to say that in the Highway Department alone there are fully 800 employees, and they do very little office work, except for a short time in the winter. They work ten hours in the field and often longer, and they do all their traveling on foot, carrying their instruments. They are a hardy, rugged lot of engineers, made up of men with all sorts of experience in railroad and heavy construction work. The engineers from the state engineer's department are also used to hard work and long hours, except those that have always been in the office, of whom there are comparatively few. In this department you will also find men with a variety of experience.

I cannot see where you can find a better set of engineers for military duty than those employed by the State of New York, and the majority of them are willing to serve—but not as the proposition was put to them by the governor. His plan would make them lose their salary and vacation time, as well as pay their own expenses while training. In the Highway Department most of the men have families and cannot afford to lose any time, as a large number of them are laid off for a month during the winter through lack of work.

Hudson, N. Y., Feb. 25, 1916. O. HASBROUCK.

Sir—I was much interested in reading a letter in *Engineering News*, Feb. 24, by so-called "Vet," concerning the making of military engineers out of the engineers in the employ of New York State. The statements contained therein are far from the truth, and therefore, in justice to the many able and faithful engineers in the employ of New York State, I have considered it unwise to let the letter pass unchallenged.

In the first place, I have grave doubts whether the one who signs himself "Vet" has an intimate acquaintance with a very large proportion of the 1,200 engineers in the state employ. If he has such acquaintance, his ability to judge human nature, especially as found in engineers, is admitted only by himself. He says that a large proportion of the engineers in the state employ have never done any work independently. A great deal of evidence would be necessary to establish this contention. I will say that such a broad statement is not in the light of reason nor founded on facts. The actual value of construction work supervised by the state's engineers has long since passed the $100,000,000 mark. Consequently, the statement that they have always worked inside an office is simply absurd.

As for the test of driving a nail through a block, whether the block would be split in the operation would depend upon the size of the nail as well as upon that of the block.

A day when men in the various construction parties on the state canals and highways do not walk more than ten miles over rough land is most uncommon. Whether or not the New York State engineers know the exact weight of a rod of barbed wire is immaterial to the discussion of so broad a subject as their fitness for military service. A great deal would depend upon the extent of the barbed-wire entanglement to be placed, as well as upon the number of men included in a handful, to decide whether an equivalent number of men of "Vet's" type could do the work faster.

The New York State engineers will be ready when their services are required. All those whom I have been in contact with have appreciated Governor Whitman's motive and have filed their applications for the purpose of entering the training camps under the instruction of United States Army officers. A. E. STEERE.

Mark Building, Amsterdam, N. Y., Feb. 25, 1916.

Some Faults of Our Patent Laws

Sir—Your article on Bone's retaining-wall patent and your further remarks on reinforced-concrete patents, in your issue of Feb. 10, open up important matters for discussion. The following quotation is especially pertinent:

> In many patents the engineering evidence of use prior to the patent application is overwhelming, though there is much difficulty in impressing such evidence on the intelligence of the court.

If such evidence is overwhelming, why was it not considered by the Patent Office? The answer is simply this: The entire appropriation for investigation of prior public use made by the United States Congress amounts to the paltry sum of $150 per year for all patents and all arts. Likewise, the appropriation for proper filing systems of technical literature and facts bearing upon the allowance of claims is equally insufficient. The result is obviously this: Patents are issued embodying claims based upon an insufficient consideration of the prior art and such that, if the art, as subsequently indexed and tabulated by the industrious examiners of the Patent Office at a later date, were to be considered, these claims would be held invalid.

Under these circumstances why should there not be a provision similar to that of Canadian law for the annulment of patents erroneously issued? This would prevent the levying of legalized blackmail on the public at large upon the basis of mere paper patents.

The difficulty you mention of impressing engineering evidence on the intelligence of the court is illustrated in a recent decision in which the judge held that a patent on reinforced concrete to be valid must cover the process of burying metal in concrete—a process older than the Christian era. In another case the judge asked whether a patentee who applied for a patent in 1901 was the first ever to have embedded metal in concrete.

It is frequently said that the majority of judicial decisions in the field of structural engineering are rendered contrary to, rather than in accord with, the fundamental

laws of physics and mechanics. This is because of the mischievous success of patent attorneys whose highest ambition is to mislead the court by plausible misrepresentation of these principles. Many users are coerced into paying tribute upon an inoperative patent, because of the expense of a contest and the general feeling that in trying a case before a tribunal of judges innocent alike of any knowledge of the principles of mechanics and the history of the prior art the claims in a technically worthless patent have just as much chance to be upheld broadly as the claims upon a useful improvement.

In this country, unlike Canada, there is no legal machinery available to curtail the fraudulent claims of swindlers and speculators in the patent field. The judicial outcome of a patent case, after rendering a decision in the absence of a complete exposition of the prior art, permits a kind of legalized blackmail not tolerated in countries more advanced in the scientific administration of justice than our own. Is it not time that some effort was made toward improving this condition? Would it not be a distinct advance for the court to requisition from the Patent Office the latest complete prior-art records and the primary examiner's review thereof, so that the deficiency due to lack of proper appropriations in the past may be remedied? C. R. SPEERS.

Des Moines, Iowa, Feb. 22, 1916.

Initial Stress in Rivets

Sir—Mr. Friedland's article on rivet tension, in your issue of Feb. 17, contributes a most ingenious analysis to the literature of riveted joints. This analysis is based upon the assumption that we have elastic action in the shank of the rivet. I should expect, in view of the hundreds of degrees of cooling of the rivet after driving, to find the shank in tension up to the elastic limit. Any increase of this tension would cause stretch in the shank and a consequent relief of the initial tension. It seems probable that the rivet shank could endure a permanent deformation that would make the initial shank tension zero and leave as stress in the shank only the load pull on the head.

It is certainly conservative to say that rivet shanks "often have an initial stress above what would be permitted in safe steel design," but the conclusion therefrom that tension rivets are to be avoided is unwarranted.

Lafayette, Ind., Feb. 19, 1916. ALBERT SMITH.

A Tip to Manufacturers

Sir—A great amount of money is spent each year by manufacturers and sales corporations for trade-paper advertising, house organs, catalogs and other sales literature that does very little of the work for which it is designed. A large percentage of such unavailing literature finds its way to the waste basket at once, while some that on casual observance seems good enough to keep is consigned to the same place the first time the recipient chances to use it as a source of information on the subject to which it pertains. This is true to some extent (although not as much so as in the past) of literature on materials, both "finished" and "raw," entering into the construction of buildings.

Some catalogs and trade publications give a vast amount of information regarding the product or products in question, so that these articles can be properly and satisfactorily applied; but others speak in flowery-worded phrases of the great number of buildings in which a certain material has been used, of "repeat" orders, etc., all intended to make the prospective purchaser, architect or engineer feel that a special privilege is being granted him as a member of the *élite* in return for his using the material in question. As a general rule the purchaser, architect or engineer does not care so much about the photographs of buildings wherein a certain material or product has been used as he does about general and typical citations serving as a guide to his judgment.

As an actual example of the uselessness of some catalogs, the following experience of the writer is typical of several fruitless searches through such publications for data of a useful character. In designing a machine shop it was found advisable to provide ventilation in the roof by means of circular galvanized-iron or copper ventilators. A search through the various catalogs and trade publications failed to disclose any real, pertinent information on the subject of the proper size of ventilators to be used for different classes of occupancy. Much space was devoted to photographs of successful installations, but not enough information regarding the general conditions to be met was given to allow a designer to gain any idea as to the proper size to be used. This could readily have been done by means of a small cross-section of the building, showing type of construction, height, occupancy, side-wall ventilation, and spacing of circular vents in the roof.

If each manufacturer would do this, a designer would have some basis for comparing the relative merits of the different brands; and satisfied, instead of dissatisfied, customers would be the result in the great majority of cases. A firm with a good product should not be at all backward in citing the facts that the users should have before them in order that an intelligent selection can be made.

Other examples of similar nature could be cited; but in order to make this letter brief, they are omitted. When expending large sums of money for catalogs, it seems like short-sighted economy to save money by omitting such data as require some labor and skill to prepare, but which constitute a vital factor in a successful selling campaign. Too many catalogs are mere picture books that cost a lot of money and say practically nothing regarding the product in question. As a result they are poor business-getters. C. A. TALOGS.

Chicago, Jan. 27, 1916.

NOTES AND QUERIES

George Herrick Duggan is president of the Canadian Society of Civil Engineers. In the biography published in "Engineering News" of Feb. 17 Mr. Duggan's middle name is incorrectly given as Eric.

Responsibility for Damage to a Transit—An assistant in a city engineer's office was running the center line of a public street, using a transit belonging to the city engineer. There was plenty of room for teams to pass the transit, and several teams had passed where the work was in progress. A buggy carelessly driven came past, however, knocked the transit over and broke it. Inquiry is made as to whether the party causing this damage can be made to pay for the cost of repairing the instrument. Offhand, one would say that it is the duty of the instrument man to guard his instrument, but there may be cases where courts have ruled the contrary.

Arkansas and White River Floods of 1916

By Elliott J. Dent*

In the drainage basin of the Arkansas River above Fort Smith there were two storms during January, 1916, and two distinct rises in the river at Fort Smith. In the lower river the two rises were merged into one. At Fort Smith the peak of the first rise passed on Jan. 23 with a gage height of 26 ft., while the second rise did not begin until Jan. 27. Farther downstream, at Little Rock, the crest of the first rise passed on Jan. 26, and the second rise began on Jan. 28. The net result was a great flood in the lower Arkansas River, although greater floods are of record.

In January, 1916, the crest of the flood from the White River above the mouth of the Black had passed the junction before the crest of the flood from the Black River reached that point.

With the Mississippi at a high stage, as was the case during the period under consideration, the flood heights in the lower Arkansas and lower White River Valleys are materially affected by the backwater from the Mississippi. In the Arkansas River this effect is felt well above Red Fork, and in the White River it reaches above Clarendon.

A discussion of the floods of the lower Arkansas and White River Valleys is somewhat involved, owing to the many factors entering into the case, but for the upper rivers we have the simple relation between rainfall and floods shown by the accompanying table.

RAINFALL AND RIVER STAGES AT FORT SMITH ON THE ARKANSAS RIVER AND CALICO ROCK ON THE WHITE RIVER

	Fort Smith		Calico Rock	
Jan., 1916	Rainfall, In.	Gage Height, Ft.	Rainfall, In.	Gage Height, Ft.
21	0.78	10.2	0.24	5.3
22	0.00	24.8	0.30	6.3
23	0.00	26.0	0.00	7.3
24	0.00	25.5	0.00	8.1
25	0.00	24.0	0.00	7.9
26	2.50	21.6	0.00	6.7
27	0.19	29.0	1.02	8.8
28	0.35	30.0	0.35	22.2
29	1.02	30.6	1.30	32.5
30	1.05	32.7	1.80	36.6
31	T	32.0	0.90	48.5

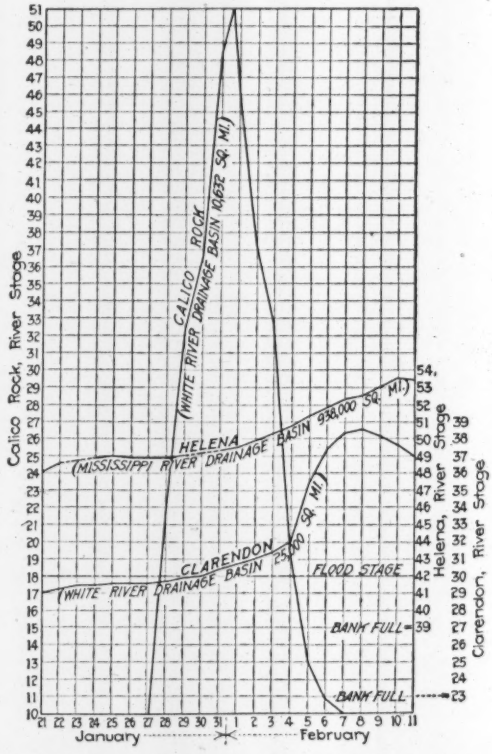
At Calico Rock the highest stage reached was 51 ft. at 9 p.m., Jan. 31, the highest previously recorded stage having been 48.5 ft. In the White River new records have been established throughout the valley below Calico Rock. A diagram showing the river stages at Calico Rock and Clarendon on the White River and at Helena on the Mississippi River is shown herewith.

Along the banks of the Arkansas and White Rivers the levees of the Mississippi River system extend but a short distance upstream from the junction point. Above the ends of these works local improvement districts have in many cases built levees. The latter were in several instances overtopped or broken. Where no levees have been constructed, the damage due to an extra foot or two in the height of the flood above the ordinary flood stage is relatively small. Where levees have been built, however, an extra foot, by overtopping and breaking the levee, may cause the submergence of large areas to considerable depths when no submergence whatsoever has been anticipated.

The City of Newport, on the White River, was guarded by a local levee. When the reports of the magnitude of the flood in the upper river were received, it was at once apparent that the levee would be overtopped and the city flooded. Preparations were promptly made, and everything movable was placed above the flood level. The result was that the loss was reduced to a minimum.

In the City of Clarendon it was, until too late, thought that the local levee would protect the city. The loss there was considerable; but as numerous water craft were available, it was possible to remove a large percentage of the population before the shortage of supplies became acute.

There were no breaks in the levees on the north bank of the Arkansas River, but breaks of considerable magnitude occurred on the state convict farm at Cummins and at other places on the south bank. Water flowing through these crevasses did not return to the Arkansas River, but flowed south across country into the bayous heading near the Arkansas River and emptying into the Ouachita.



RIVER STAGES AT CALICO ROCK AND CLARENDON ON WHITE RIVER AND HELENA ON MISSISSIPPI RIVER

Farms and towns in this locality had long been free from floods. The overflow came suddenly, and people as well as stock were marooned in houses or on small knolls or ridges, in many cases without adequate supplies or shelter. River boats could not go to their rescue, and flat-bottomed rowboats built of material at hand were the only means of transportation. Assistance was promptly rendered by the communities that were not flooded, and the situation was well in hand within four or five days.

The interruption to railroad traffic was considerable, but was principally due to the submergence of tracks. No large bridges were lost, and the washouts were less extensive than had been feared. It is doubtful whether in the Arkansas and White River Valleys there was a

*Major, Corps of Engineers, U. S. A., Little Rock, Ark.

single line 200 mi. or more in length that was not blocked at some time during the high water.

By Feb. 11, nearly all lines were operating, although fast trains were not maintaining their schedules. The principal exceptions were the lines affected by backwater from the Mississippi. The main line of the Cotton Belt system was still interrupted at Clarendon, and the Missouri Pacific line from McGehee north to Helena and Memphis was under as much as 10 ft. of water at the worst point.

As stated in the telegraphic report printed in *Engineering News* of Feb. 10, about 1,500 lin.ft. of revetment was lost at Pine Bluff. A few small dwellings and other structures caved into the river as a result of this failure. The maximum depth of the caving measured perpendicular to bank line was about 150 ft. As the water receded, it became evident that there had been some material changes in the channel at Little Rock, and a considerable expenditure for bank protection on both sides of the river may be necessary in that vicinity.

Lock and dam No. 2, White River, were considerably damaged. The lower lock gates were carried away, and there were several minor washouts in the levees at each end of the dam. The top timbers of the dam were carried away in two places, but the full extent of the damage cannot be determined until a lower stage is reached. Unless the breaks are materially enlarged before the next low-water season, the only interruption to the use of the dam will be the time required to replace the lower gates.

Purchasing Water-Works Plant of Bath, Maine

By Frederic H. Fay*

In the recent acquisition by the City of Bath, Me., of its water-works system formerly owned by a private company two precedents have been established so far as important privately owned plants in the East are concerned. The first is that in the determination of the price to be paid by the city for the works the whole matter was left to two engineers—Frederic H. Fay, of Fay, Spofford & Thorndike, Boston, and William B. Getchell, Augusta, Me.—and they were able to reach an agreement without calling in a third arbitrator. The second precedent is the fixing of an award that is less than the reproduction cost of the plant.

The award of the appraisers in the Bath valuation case is $539,500, which is nearly $20,000 below the reproduction cost agreed to by both parties. Notwithstanding that this award is considered a very favorable figure for the city, the water company is apparently satisfied with the result, especially when it is considered that the expense of costly litigation has been saved, which might easily have cost the two contestants $50,000 apiece. Moreover, a large amount of time, valuable to both parties, but ordinarily taken up in legal proceedings, was saved. From the time when the Water District was incorporated and began preparations for the purchase until the final payment was made, less than four months elapsed.

The Bath Water District was created under Chapter 197 of the Private and Special Laws of Maine of 1915, accepted by vote of the inhabitants of Bath. The date of incorporation of the district was Nov. 1, 1915, when the three trustees who are to administer the affairs of the district took office. Immediately thereafter the trustees engaged Fay, Spofford & Thorndike as consulting engineers to the district. During November and December this firm made an inspection and appraisal of the Maine Water Co.'s plant at Bath, in which work representatives of the company coöperated to the fullest possible extent on facilitating the inspection and furnishing data of the system. In fact, throughout the whole proceedings the relations between the company and the city and the representatives of both have been most friendly and mutually helpful.

During the month of January, 1916, an understanding was reached between the company and the city, by which it was agreed that a part only of the questions at issue—the value of the water rights and the reproduction cost and depreciation of the physical plant—should be referred to the engineers representing the two parties, in the hope that these items might be mutually agreed to and finally disposed of before litigation was begun, the agreement of the engineers upon any or all of these items to be binding upon both parties. Under this arrangement a conference of engineers was begun on Jan. 31. The Maine Water Co. was represented by three engineers—William B. Getchell, Augusta; John H. Burleigh, Waterville; and Walter F. Abbott, Bath (the latter being superintendent of the company's Bath plant)—while negotiations in behalf of the city were conducted by Frederic H. Fay, for the consulting engineers, Fay, Spofford & Thorndike, assisted by E. L. Walker, C. A. Farwell and R. W. Horne, assistant engineers for that firm. In three days' conference these engineers reached an agreement on a reproduction cost as of Jan. 1, 1916—the date of the taking—of all parts of the physical plant as well as the value of the water rights. They failed to agree, however, upon the subject of the depreciation of the physical plant.

At the conclusion of this first conference the novel suggestion was made by the counsel of the water district to the president of the company that the entire matter of the determination of the purchase price be left to two arbitrators—Mr. Getchell and Mr. Fay—with the proviso that in case these two engineers failed to come to an agreement they should select a third arbitrator, not an engineer, the decision of these arbitrators to be final and binding upon both parties. Messrs. Fay and Getchell met in conference on Feb. 10, 1916. Through their knowledge of the principles governing engineering values and their intimate acquaintance with the facts of this particular case, and without even the necessity of calling in a third arbitrator, they reached an agreement that day and made their award of $539,500 as the purchase price for the entire water-works.

There are two distinct methods of conducting negotiations of this nature—one, the method of contention and litigation, in which the interests of the parties are regarded as diametrically opposed; the other, the method of confidence and coöperation, in which the community of interests of the parties is given full recognition. In the minds of many people, especially in the past, the first method has been regarded as the only one possible. The purchase of the Bath water-works furnishes an excellent instance of the uptodate efficiency of the latter method and of its practical applicability as a saver of money, time and other valuable assets.

*Of Fay, Spofford & Thorndike, Consulting Engineers, Boston, Mass.

News of the Engineering World

Road Builders Association Meets at Pittsburgh

The American Road Builders Association held its thirteenth convention at Pittsburgh last week, with a total registered attendance of about 2,000. An exhibit of road machinery and materials was arranged in the old exposition building, with some 80 exhibitors. The recently elected president of the association, Col. E. A. Stevens, State Highway Commissioner of New Jersey, presided at the sessions. The States of New Jersey, Pennsylvania and Ohio were especially well represented in the attendance.

The sessions held on Monday evening and Tuesday morning were devoted to the popular side of good road work, addresses being made by Governor Major and Congressman Shackleford, of Missouri, the latter the author of the Federal aid bill that has passed the House and is now before the Senate; also by Hon. Philander C. Knox, ex-Secretary of State, the Mayor of Pittsburgh and other public officials.

In the technical sessions a paper by R. Keith Compton, chairman of the Baltimore Paving Commission, urged that cities should have legal authority to prescribe the type of construction to be adopted for street-railway tracks laid in paved city streets. This has been done in Baltimore; and on heavy-traffic streets the railways are required to lay ties on a concrete platform. The space between the ties is afterward filled in with concrete up to the level of the under side of the paving blocks. These latter are laid on a bed of cement and sand and not on sand alone.

In order to lay this track without interrupting traffic while the concrete is setting, the track is laid on a bed of broken stone under the ties. It is then poured full of cement grout of the consistency of thin cream; and it sets satisfactorily, notwithstanding the continual passage of cars over it while it is setting.

Several electric-railway engineers present sharply criticized the Baltimore requirement, arguing that the rigid concrete bed was too hard and had proved unsatisfactory, as had the earlier attempts to lay the rails directly on concrete beams. They preferred to lay the ties on broken stone, which could be tamped when necessary to keep the track in surface.

George E. Datesman, Director of Public Works of Philadelphia, read a paper describing the control of street openings in pavements in that city. He claimed that Philadelphia had a more complete system of records of its underground piping than any other great city of America and described in detail the methods of control and record.

H. W. Durham, late chief engineer of street paving of the Borough of Manhattan, referred to the claim often made that European city streets are not torn up for pipe connections as are American streets and said that his own investigations in foreign cities proved that street openings in pavements are as numerous there as here.

W. D. Uhler, Chief Engineer of the Pennsylvania State Highway Department, described the new concrete road between Easton and Allentown, now under construction, of which about 3½ mi. was completed last fall. Tests made to determine how long a time the concrete should be mixed in a batch mixer show that the time should be from 1½ to 2 min. Test cubes of concrete mixed 3, 4 and 5 min. did not show materially greater strength. Transverse joints are spaced 35 to 40 ft. apart, and metal joint plates have been used on only 1,000 ft. Reinforcement is used and is delivered in flat sheets instead of in rolls. It is placed 2 in. below the top of the slab. So far only two small cracks have showed up in the entire 3½ mi. The concrete is 6 in. thick at the sides and 8 in. at the crown, with a 1 to 2 to 3 mix, and is costing $1.20 to $1.25 per square yard.

The brisk discussion of Mr. Uhler's paper evidenced the great interest in concrete-road construction at the present time and brought out some notable examples of concrete paving that has been down for many years and is still in excellent condition.

The gravel roads built by the State of New Hampshire were described by F. E. Everett, State Highway Commissioner. These roads, built at an average cost of less than $3,000 per mile, have greatly pleased the summer-tourist traffic, which they chiefly serve. They were described in *Engineering News,* Dec. 9, 1915, p. 1110.

"Brick Streets and Roads" was the subject of a paper by H. E. Breed, Deputy Highway Commissioner of New York. After describing the present standard practice in brick-road construction in New York, he said that the state has now 290 mi. of brick roads, which have cost on the average $2.01 per square yard. The average maintenance cost of these roads last year was $176 per mile, but this included some extensive repairs on some 10-yr. old roads that had not been properly built and maintained. Excluding this work, the cost of maintenance was only $131 per mile, which included work on ditches, drainage, railings, etc., apart from the road surface.

That stone-block paving practice is tending toward the use of shallower blocks, only 3 to 4 in. deep, was the contention of Charles F. Knowlton, of Boston, in a paper on "Recent Tendencies in Granite Paving." This statement was denied by H. W. Durham, who pointed out that the increase in labor costs in laying small blocks tends to offset other saving. Others claimed that a deeper block is needed to give sufficient bonding surface for the joint material.

R. B. Sage, Chemist of the New Jersey Road Department, discussing the function of stone in a bituminous-concrete pavement, urged that the stone chosen should have low absorptive power. Otherwise, water soaks through the stone to the asphalt matrix in which it is held, and the stone is thereby loosened. He also urged the use of large stone in the mixture to reduce voids.

Other papers read were by Joseph W. Hunter, on "Road Foundations"; Henry L. Bowlby, on "The Columbia

Highway"; and Charles N. Upham, on "The Du Pont Road."

The Committee on Resolutions made a report at a business session on Thursday afternoon, and the following were adopted:

Resolved, That this association invites the attention of State, county and municipal authorities to the value of traffic censuses as a means of determining the relative values of various forms of pavement and road construction and would strongly recommend that systematic traffic censuses be taken from time to time on all classes of roads and pavements in order that more definite knowledge of the economic value of the different types of construction may be obtained. In taking such censuses separate counts should be made of horse-drawn vehicles, light and heavy, and of motor vehicles, classified as pleasure and commercial cars, with notes as to the use of especially heavy trucks and vehicles with tires especially injurious to roads. Where it can be used, in order to obtain comparable information, the standard form for taking traffic censuses reported by a committee of the American Society of Civil Engineers is recommended.

Resolved, That where funds for road construction are raised by an issue of bonds, this association urges that the serial form of bond, by which a part of the principal is paid off each year, be adopted in preference to bond issues to be paid by accumulations in a sinking fund.

Resolved, That this association urges that laws be passed and enforced in each state establishing reasonable limits for the weight, dimensions and speed of heavy motor vehicles.

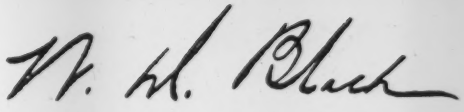
Col. William M. Black To Be Chief of Engineers

Col. William M. Black, New York City, has been appointed Chief of Engineers to succeed Gen. Dan C. Kingman, who retired from office on Mar. 6, having reached the age limit. Colonel Black is one of the best-known officers in the Corps of Engineers. He was born at Lancaster, Penn., Dec. 8, 1855, and can serve as Chief of Engineers, therefore, for nearly four years before reaching the age limit.

Colonel Black received his early education at the old Franklin and Marshall College in Pennsylvania and graduated from West Point in the class of 1877. During his long time of service he has had an unusual amount of experience in municipal engineering work. He was an assistant to the late Col. William L. Ludlow in the early 80's, when the colonel "cleaned up" the Philadelphia Water Department. In 1897 and 1898 he was Engineer-Commissioner of the District of Columbia, and he rendered important service again under General Ludlow in the sanitary regeneration of Cuba and its cities after the Spanish-American War.

During the past seven years Colonel Black has been in charge of the United States Engineer Office in New York City, where many important municipal problems have come before him, such as the investigation in connection with the pollution of New York harbor by sewage discharges.

Colonel Black rendered important service during the Spanish War, being successively Chief Engineer of the Third Army Corps, Chief Engineer of the Fifth Army Corps and on the staff of the Major General in Command during the last half of 1898. He served at Tampa, Fla., and organized a battalion of engineers for the Porto Rico expedition and bought and had charge of all the engineering material used. He was in command of the first landing party of United States troops at Guanica, Porto Rico, July 25, 1898. Colonel Black was the first United States engineer officer to take charge in Panama when the United States took over the Isthmian Canal work.



Colonel Black has been for some time the senior member of the Board of Engineers for Rivers and Harbors and also Department Engineer of the Eastern Department.

One of the most notable engineering works with which Colonel Black's name is connected was the removal of the wreck of the battleship "Maine" in Havana harbor. He was a member of the Board of Engineers in charge of this work and gave to it his personal supervision.

Denver Water-Works Struggle

Promise of settlement of the long struggle between the City of Denver and the Denver Union Water Co. is afforded by an option contract for the purchase of the works by the city, entered into between the Public Utilities Commission of the city and the Denver Union Water Co. on Feb. 21. The agreement has also been approved and adopted by the Denver City Council and by the stockholders of the Denver Union Water Co.

Under the option contract both parties accept $13,416,000 as the value of the entire holdings of the company used to supply water to Denver. This valuation is as of May 1, 1914, and is subject to a depreciation charge of $140,000 per annum on the one hand and to the cost of betterments since May 1, 1914, on the other. The

basic sum named is the valuation placed upon the plant by W. J. Chinn, special master, appointed by the Federal District Court in a suit brought by the company to prevent a 20% city-ordinance reduction in the rates charged by the company. This valuation was fixed by Mr. Chinn some months ago (see *Engineering News*, Oct. 21, 1915, p. 814) and was confirmed by the Federal District Court on Jan. 28, 1916. In accordance with the option agreement the case is to be appealed to the Supreme Court of the United States. Each side will exert every possible effort to end the suit upon its merits. The option contract holds good until twelve months after the United States Supreme Court shall have disposed of the case. No betterments are to be made to the plant after Feb. 21, 1916, without the approval of the Public Utilities Commission. When the transfer is effected, the price of the works as finally determined by adjudication is to be met in 4½% bonds or in cash, at the option of the city. The city agrees that during the life of the option it will refrain from constructing a duplicate plant, but it is free to complete its surveys now in progress for new work.

There has been almost constant water-works turmoil at Denver for at least a quarter of a century. In 1891 the Citizens Water Co. built works that paralleled the mains of the Denver City Water Co. A bitter rate war ensued. The two companies were consolidated in 1894 as the Denver Union Water Co. Since then there has been much litigation between the company and the city. In 1909 a valuation of $14,400,000 was placed upon the property of the Denver Union Water Co. by a commission of engineers consisting of M. L. Holman, John R. Freeman, C. L. Harrison, Allen Hazen and Frederic P. Stearns, the latter agreeing to the valuation except as regards the item for water-rights.

In 1910 a Public Utilities Commission was created by popular vote, primarily for the task of establishing municipal ownership of water-works. The commission was instructed to offer the Denver Union Water Co. $7,000,000 for its plant. It was also empowered to build works. The company refused the offer and brought suit to prevent the city from going on with construction. In 1913 the United States Supreme Court decided this suit in favor of the city. In January, 1914, a committee appointed by the Public Utilities Commission valued the property of the company at $10,045,000. The valuation dealt with the physical plant only, except for an allowance of $375,000 for development expenses. Other elements of the property were not taken into consideration in view of points of legal conflict.

A. Lincoln Fellows, of the Field, Fellows & Hinderlider Engineering Co., is president of the Public Utilities Commission and was also a member of the committee that placed on the plant the valuation just mentioned.

Regulating Engineering in the State of Pennsylvania

The Commission of Engineers, which has for nearly three years been investigating the question whether engineering practice in Pennsylvania should be regulated by law, has presented a report to the governor, which was made public on Mar. 2. The commission does not recommend legislation requiring all engineers to pass examinations and be registered in order to practice their profession. It does recommend that there should be regulation by law of the practice of engineering in the state and municipal service. The plan it has worked out is, in brief, that engineers in subordinate positions should be required to pass civil-service examinations. Engineers above a certain rank should be selected on their professional record without examination; and consulting engineers and experts for temporary engagements should be required to file with a state board a sworn statement of their professional record. The final conclusions of the commission are as follows:

1. It is desirable in general, for the better protection of the life, health and property of the citizens of this Commonwealth of Pennsylvania, that the employment, retention and discharge of engineers, surveyors and technical men in public service, where competency may be readily determined by examination, should be subject to state or municipal civil-service regulation.

2. In cities of the first and second class in Pennsylvania such civil-service examinations may properly be conducted, as at present, under independent local management; but consulting engineers and experts for special temporary engagements should be exempt from examination.

3. The above class of engineers and experts should be required to register with a state board before being permitted to practice. They should file a sworn statement of their professional record and confirmatory statements from others, if called for, and unless promptly disapproved, this filing should be sufficient precedent to practice. The board should have power, for adequate cause, after hearing, to cancel the registration. Also to issue certificates to such experts, if desirable.

4. In the other cities (except cities of the first and second classes) and in the counties, boroughs, townships and municipalities of Pennsylvania and for state work, it is desirable that such civil-service examinations, mentioned in paragraph 1, be conducted by a state department, board or commission in order that these examinations may be governed by reasonable and consistent standards and that a single body may be charged with the enforcement of such standards.

5. Engineers, surveyors and technical employees of the state, counties and municipalities, other than first- and second-class cities, excepting those in administrative positions, should be required to qualify before the said department, board or commission by methods discretionary with the said board. The determinations of standards and classifications should likewise be in the hands of the board; but appointments to engineering or technical positions above a certain rank should not be governed by competitive civil-service examinations. These appointees should be chosen on the basis of their professional record.

6. It is desirable that such appointments to positions of high rank should be made at the initiative of and by the state or municipal department or officer seeking to have the position filled; but with the approval of the State Civil Service Examination Board.

7. This employment will be governed by the same principles as under private corporate management. All those who have passed such examinations should be given certificates of fitness for such work as they can do.

8. Other engineers in practice, who may desire, should have the right to be examined and secure a certificate of fitness and competency in their particular line or specialty.

9. The exemption of public-service companies from the provisions of civil-service regulation at this time is suggested with the thought in mind that a beginning having been made with public works, erected, operated and maintained by the state and municipal governments, prudence dictates a reasonable period of time in which to observe the working out of the system before any extension of the system be considered.

10. Plans and specifications for public works built or to be built, operated and maintained by public-service companies and municipalities, should, under the provisions of existing laws, be handled in the proper and respective departments of the state government.

11. It is desirable that the State Civil Service Examination Department, Board or Commission should be authorized to investigate charges of incompetency or official dereliction on the part of engineers or technical men appointed through civil service (excepting cities of the first and second classes) and that it alone should be empowered to disqualify such public servants, and only for adequate cause.

12. It is further desirable that the said State Civil Service Examination Board should act in an advisory capacity in connection with the employment of consulting engineers or experts for special temporary engagements and that no such engagements should be legal if made against its disapproval.

It is the prescribed duty of the Engineers' Commission "to recommend to the governor such measures as, in their judgment, should be enacted into laws." It was the evident intent of the General Assembly that the commission should not be limited by narrow bounds when it came to presenting recommendations to the governor.

There is now an Economy and Efficiency Commission of the Commonwealth of Pennsylvania investigating the duties of every person in the employ of the state, to ascertain what changes, if any, may be necessary to secure greater uniformity, economy and efficiency in the work of the various departments, commissions, bureaus, boards of trustees and other persons in charge of institutions owned or controlled by the state and in the methods of disbursing and accounting for state funds. It is among the possibilities that this Economy and Efficiency Commission may work out the details of a former suggestion that the scope and the duties of the Board of Commissioners of Public Grounds and Buildings shall be extended to really comprise a State Board of Public Works. This department now provides the quarters and furnishes supplies to the several state departments. It inspects the erection of state buildings, of county bridges over navigable streams and has charge of the Capitol grounds and buildings.

If this department were to be reorganized into a Board of Public Works with several bureaus, one of which would do all the architectural work for state buildings, another all construction engineering work on state buildings and institutions, another do all the purchasing of supplies for each and every state department and institution, another have overseeing charge of all state ground and property, etc., etc., it might be very pertinent to inquire into the economy and practicability of adding to the department a bureau of State Civil Service Examinations. The economy would be in the supervision of the work by the Board of Public Works in joint occupancy of office space and fittings and furniture and in the routine maintenance of the bureau.

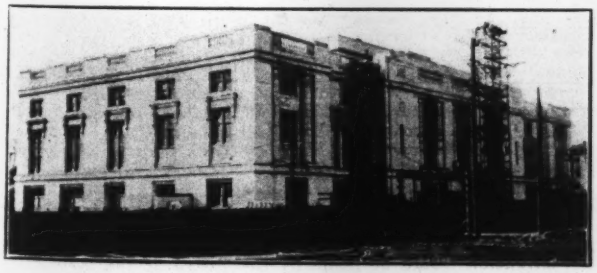
Somewhere in the scheme of state government there should be a Civil Service Examination Department, Board or Commission. The Examining Board should be composed of engineers having the qualifications demanded of the highest-grade membership in the national engineering societies. The members of this Examining Board might be drawn from the staff of the several state departments and from among professional engineers generally. The latter should be fully compensated for their services.

With these suggestions the Engineers' Commission recommends for your consideration the enactment into laws of the measures embraced in paragraphs 1 to 12 inclusive.

F. HERBERT SNOW, Chairman; JOHN PRICE JACKSON, Secretary; SAMUEL A. TAYLOR, G. H. WEBSTER, J. MURRAY AFRICA.

Dallas Union Station To Open May 1

The Dallas, Tex., union railway station, described in detail in *Engineering News*, June 25, 1914, is nearing completion. The accompanying view was taken Feb. 12. It is stated that the station will be ready for use by



THE DALLAS (TEX.) UNION DEPOT AS IT APPEARS TODAY

May 1. The total cost of the improvement will be about $5,000,000. The station will serve seven railways—the Gulf, Colorado & Santa Fe; the Chicago, Rock Island & Pacific; the St. Louis & San Francisco; the Missouri, Kansas & Texas; the Texas & Pacific; the Houston & Texas Central, and the St. Louis Southwestern Ry. lines. This is not a terminal station, the tracks running through.

The chief engineer of the Dallas & Terminal Railway Union Depot Co. is C. H. Dana. The general contractor is C. W. Thompson, Chicago, and the local subcontractor, the Hughes & O'Rourke Construction Co.

Dayton Flood-Protection Plans Ready for Adoption

Five immense reservoirs designed to afford protection against floods 40% greater than the Dayton flood of 1913 and river-channel improvements in nine cities—all at an estimated cost of $23,539,872—are the outstanding features of the plans filed at Dayton, Ohio, on Feb. 29 by Arthur E. Morgan, chief engineer of the Miami Conservancy District. The plans will be open to public inspection for over a month, after which it is expected

TABLE 1. DIMENSIONS AND CAPACITIES OF MIAMI CONSERVANCY DISTRICT RETARDING DAMS AND RESERVOIRS

	Germantown	Englewood	Lockington	Taylorsville	Huffman
Average elevation bed of river	723	768	876	659	777
Average elevation valley floor	730	784	885	770	785
Elevation of spillway crest	815	876	938	818	835
Elevation of top of dam	830	892.5	954	837	850
Maximum elevation of water surface for a flood like 1913	805	862	936	815	830
Area submerged in a flood like 1913, acres	2,950	6,350	3,600	9,650	7,300
Maximum amount of water stored in a flood like 1913, acre-ft	73,000	209,000	63,000	152,000	124,000
Maximum average depth of water in reservoir in a flood like 1913, ft	24.8	32.9	17.4	15.7	17.1
Length of crest of dam, ft	1,210	4,660	*6,400	*2,980	*3,340
Thickness of dam at average elevation of valley floor, ft	650	740	410	390	380
Length of outlet conduit, ft	520	725	45	40	40
Number of conduits	2	2	2	4	3
Total sectional area of outlet conduits, sq.ft	182	214	158	1,116	705
Maximum discharge of conduits in 1913 flood, sec.-ft.	9,340	11,000	8,630	51,300	32,600
Length of spillway crest, ft	55	100	72	132	100
Vertical diameter of conduits, ft	12.3	13.5	10.0	20.5	17.6
Yardage of earth in dams	865,000	4,000,000	1,135,000	1,235,000	1,655,000
Yardage of concrete in dams and appurtenances	20,000	38,000	38,000	47,000	40,000

* Including spillway crest.

TABLE 2. ESTIMATED COST OF MIAMI CONSERVANCY DISTRICT PROJECT

Retarding Basins:	
Germantown	$640,235
Englewood	1,959,632
Lockington	865,218
Taylorsville	1,508,674
Huffman	1,299,125
	$6,272,884
Local Flood Protection:	
Piqua	$128,801
Troy	1,383,619
Dayton	1,383,619
West Carrollton	22,416
Miamisburg	167,267
Chautauqua	4,283
Franklin	115,076
Middletown	81,305
Hamilton	1,274,962
	$3,335,303
Construction of works included in the plan	$9,608,187
Public-service relocations and damages	2,309,315
Real-estate purchases and easements	5,664,200
Administration and general expense	1,758,170
Taxes and special assessments on lands	200,000
Total	$19,539,872
Contingencies	4,000,000
Grand total	$23,539,872

that they will be officially adopted by the directors of the district.

The five dams would be built on the Miami, Mad and Stillwater Rivers and on Twin and Loraine Creeks. Their function is thus described in the engineer's report:

Each dam will have permanent openings through its base through which the ordinary flow of the river and the flow during ordinary freshets will pass unimpeded. During large floods the water which cannot pass through the outlet conduits will be held back temporarily in the basins above the dams.

The conduits are proportioned so that no more water can pass through them than can be carried safely in the improved channels through the cities below. In this manner, the runoff of a flood like that of 1913, which lasted but two or three days, would be distributed over a period of about two weeks and its height would be correspondingly reduced.

The spillway-level capacity of the five reservoirs will be 840,000 acre-ft., or 60% of the total rainfall (1,415,000 acre-ft.) of the 1913 flood. At the beginning and ending of a flood the conduits through the dams and the improved river channels will pass almost all the runoff, leaving nearly the whole reservoir capacity "available for cutting off excess flow," or "peak of the load," which is beyond the channel capacity. Continuing, the report states:

In general, the greatest rate of flow through the cities below the retarding basins in case of a storm like that of 1913 would be approximately half as great as in 1913. The channel at Dayton when improved and enlarged will have sufficient capacity to carry somewhat less than half as much water as the maximum 1913 flow, and that at Hamilton to carry somewhat more than half, the remainder in each case being cared for by retarding-basin control.

The dams will be constructed of selected and compacted earth, with cutoff trenches filled with impervious material where demanded. Each dam will have a top width of 25 ft.

The conduits for passing the regular stream flow will be on rock foundation, through the base of the dams. The spillways, which will also be on rock, will be lined with concrete, will have their crests 15 ft. or more below the tops of the dams and will be designed to "pass safely all excess water in case of a flood twice as great as that of 1913." The report adds:

It is not anticipated that these spillways ever will come into use, since to bring them into action would require a greater storm than ever has been recorded in this part of the United States. They are added as an ultimate factor of safety and would provide security in case of a flood exceeding anything recorded in the history of the country.

Table 1 shows the main dimensions of the dams and Table 2 gives the estimated cost of the reservoirs, channel improvements and other items of the project.

Inasmuch as work may be prosecuted on all the channel improvements and on most of the dams at the same time, it is believed that the whole project could be substantially completed in three years' time.

The Baltimore Sewerage Commission went out of existence on Feb. 1, through the resignation of its members on the virtual completion of its work. The sewerage and sewage-disposal system built by the commission under the direction of Calvin W. Hendrick, chief engineer, will be in charge of the Bureau of Sewers and will remain under the control of Mr. Hendrick, who now holds the position of chief engineer of the City of Baltimore.

Typhoid Fever has developed recently at Chicago and Milwaukee, both taking water from Lake Michigan. The Chicago cases are said by the Commissioner of Health to be due to surface water entering the pump pit at the 68th St. pumping station, and he has recommended that people in the southern part of the city boil the water or get inoculated against typhoid. Both cities are building new intake tunnels to take water farther from shore.

A Contractor's Magazine Blew Up in St. Louis, Mo., on Feb. 28, killing two women and injuring 64 other persons. Local estimates of property damage reach $100,000. Press dispatches state that the amount of explosives stored at the construction company's headquarters is variously estimated at from 500 to 3,500 lb., 1,000 lb. being the police record. The dynamite was to be used by the Cooney Construction Co. in building a sewer from the River des Peres to Manchester and Lanhan Ave. More than 100 glaziers were at work on Mar. 1 replacing the 5,000 windows broken by the concussion.

An Aërial Coast Patrol on the Atlantic and Pacific Coasts and on the Great Lakes is planned by a committee to be organized by Rear-Admiral Robert E. Peary. It is proposed to establish stations at intervals of 100 mi. all along the coast, each equipped with a sea-plane and operators. These sea-planes would take a position some 50 mi. off shore and patrol their respective beats continuously. It is urged that such patrols in the time of peace would be valuable in discovering wrecks, vessels in distress, etc., and that the operators thus drilled would be available for keeping watch for the approach of hostile ships in the event of war.

The Ontario Highways Act passed by the last session of the provincial legislature has recently gone into effect. Finlay MacDiarmid, Minister of Public Works, now becomes Minister of Public Ways and Highways, with W. A. McLean as Deputy Minister of Highways. Provincial aid to county road construction is increased to 40% of the expenditure for this purpose, and in addition a grant of 20% to the cost of maintenance is provided for. A lesser degree of assistance will be given to township road construction. The act also provides for provincial aid to the construction of main or suburban roads by coöperative action between municipalities acting jointly, the provincial aid to be 40% of the cost, but not to exceed $4,000 per mi.

A Break in a 12-In. Water Main in Court St., Boston, a little after 8 p.m., Sunday, Mar. 5, flooded the basements of adjoining buildings, owing to difficulty and consequent delay in shutting off the water. There are two mains in the street at this point, and the gates in the undamaged one were closed before it was found out which one had burst. So much water had escaped that difficulty was experienced in getting at some of the gates, and it was necessary to close eight of them before the flow was entirely stopped, about an hour after the break occurred. Considerable damage was done in the basements of the buildings flooded, and at noon on Monday, Mar. 6, a steam fire engine and two gasoline diaphragm pumps were still at work removing water.

A High Masonry Viaduct at Penistone, England, on the line of the Lancashire & Yorkshire Ry., failed on Feb. 3, practically without warning. According to a brief account given in "The Engineer," of London, the viaduct was 60 to 80 ft. in height and consisted of a series of short span arches carried on high stone piers. It was about 30 ft. in width, carried a double-track railway and was built 67 years ago. About the middle of January a slight crack was observed in the parapet wall, but an examination of the foundations revealed no cause for anxiety. On Feb. 3, eight minutes after a main-line train had passed, one of the arches sagged under a switching engine slowly enough so that two men on the engine had time to jump off and run to a place of safety. Two of the arches fell, and the switching locomotive, which was over the adjacent pier, slid into the chasm. An official inquiry into the cause of the disaster is in progress, and "The Engineer" meanwhile conjectures that the accident may have been due to the effect of recent rains on the pier foundations.

Rear-End Collision between a New York Central coal train and a fast freight train carrying horses killed three men and seriously injured another at 4:37 a.m. on Mar. 3, on the Livingston Ave. bridge which crosses Broadway in Albany, N. Y. The cause of the accident is stated to have been the failure of the air brakes on the coal train, while on a down grade. The heavy freight train was thrown onto track No. 1 and into the rear of the third section of train 4, which was waiting at the tower east of the bridge for a signal to enter the station yard. An express train from Albany to Herkimer was just leaving the yard westbound on track No. 2 and was sideswiped by the runaway freight train, but not seriously damaged. On the rear of the express freight were two day passenger coaches, one of steel and the other of wood with steel underframe. In the collision the two passenger coaches telescoped. There was no one in the wooden car and only four passengers in the steel car, of whom three were instantly killed. Half a dozen steel coal cars piled up and crashed into the bridge, causing severe damage to that steel structure. D. W. Dinan, Division Superintendent of the New York Central Lines, conducted an investigation.

Investigation of Sewage Disposal in Los Angeles County, California, to take a year and cost $45,000, has been recommended by the Board of Sanitary Engineers recently appointed by the Board of Supervisors of Los Angeles County. The investigation would be made by the Board of Sanitary Engineers, composed of A. H. Koebig, chairman; G. P. Robinson, secretary (city engineer's office, Los Angeles); C. G. Gillespie, sanitary engineer of the State Board of Health; A. M. Brosius and Prof. Charles G. Hyde, Berkeley. The estimate of $45,000 includes a salary of $3,000 for each member of the board; two assistant engineers at $2,500 each; $5,000 for surveyors and draftsmen; $5,000 for studies of tidal currents and $1,000 for soil analyses and a study of ground-water conditions. The two grand divisions of the investigation would be the possibilities of ocean disposal and of treatment to fit sewage for use in irrigation without nuisance.

PERSONALS

Mr. J. B. Weddell has been appointed City Engineer of Olympia, Wash.

Mr. John C. Sullivan, of New York City, has been elected President of the Old Colony Construction Co., Boston, Mass.

Mr. Albert Wolstenholme, M. Am. Soc. C. E., Superintendent of Streets of Fall River, Mass., has been made City Engineer.

Mr. Karl M. Mitchell, City Manager of Sherman, Tex., resigned on Feb. 26. He was formerly at River Forest, Ill. of River Forest, Ill.

Mr. C. A. Mees, recently Assistant Chief Engineer of the Southern Power Co. at Charlotte, N. C., has resigned to engage in private practice.

Mr. R. A. Kiefer, recently with the Gardner Governor Co. at Quincy, Ill., is now in charge of the company's New York office, located at 303 Singer Building.

Mr. N. M. Curtis has been appointed Division Engineer of the Midland division of the New York, New Haven & Hartford Railroad Co., with headquarters at Boston, Mass.

Mr. J. G. Scott, resident engineer of the Southern Power Co. at Lookout Shoals, N. C., will succeed Mr. A. C. Lee at the company's hydro-electric plant at Fishing Creek, N. C.

Mr. R. L. Pearson has been appointed Division Engineer of the New London division of the New York, New Haven & Hartford Railroad Co., with headquarters at New London, Mass.

Mr. F. H. Moyer, formerly of Pittsburgh, has been appointed Chief Engineer of the Cambria Steel Co. He succeeds Mr. E. W. Clarke, who has been made superintendent of the rolling mills.

Mr. W. W. Harmon, formerly City Engineer of Grants Pass, Ore., has been engaged by the Oregon-Utah Sugar Co. as chief engineer on the construction of that company's new factory in that city.

Mr. E. M. Confer, former Superintendent of the Coosa Portland Cement Co., Ragland, Ala., has been appointed Master Mechanic of the Marquette Cement Manufacturing Co., La Salle, Ill.

Mr. Robert Rice has been appointed General Superintendent of the Missouri district of the Chicago, Burlington & Quincy Railroad Co., with headquarters at St. Louis, Mo. He succeeds Mr. F. H. Ustick.

Mr. E. A. West, Efficiency Engineer of the Portland Light and Power Co., Portland, Ore., has been appointed to succeed Mr. Roger W. Toll (recently resigned) as Chief Engineer of the Denver Tramway Co.

Mr. F. H. Ustick has been appointed General Superintendent of the Iowa district of the Chicago, Burlington & Quincy Railroad Co., with headquarters at Burlington, Iowa. He succeeds Mr. Robert Rice.

Mr. L. K. Rourke, M. Am. Soc. C. E., of Rourke & Sherman, Consulting Engineers, Boston, Mass., has been elected Treasurer of the Old Colony Construction Co. Mr. Rourke will continue his consulting practice in Boston as heretofore.

Mr. J. E. Lawton, M. Am. Soc. M. E., for the last nine years Inspector and Chief Inspector, Panama Canal, has resigned to accept a position as Consulting Engineer and Sales Manager of Ward & Co., manufacturers' agents, Washington, D. C.

Mr. John M. Bandel, Assoc. M. Am. Soc. C. E., Assistant Engineer, Baltimore Sewerage Commission, has been appointed Engineer of the Davison Chemical Co., Baltimore. He was formerly employed by the State Roads Commission of Maryland.

Mr. Paul J. Merrill has been appointed Assistant Chief Inspector of the Dayton Metal Products Co., Dayton, Ohio. Mr. Merrill was formerly located at Dearborn, Mich., as chief civil engineer on the erection of the experimental station at Henry Ford's residence.

Mr. A. C. Lee has been made Assistant Chief Engineer of the Southern Power Co., with headquarters in Charlotte, N. C. Until recently he was resident engineer on the company's large hydro-electric plant at Fishing Creek. In his new position Mr. Lee will have direct charge of the engineering department. He succeeds Mr. C. A. Mees.

Mr. Bertrand D. Barker, Assoc. M. Am. Soc. C. E., recently Superintendent of Construction for George A. Quinlan, Assoc. M. Am. Soc. C. E., Chicago, has been appointed Assistant Chief Highway Engineer of Cook County, Ill. He was formerly for several years engineer and superintendent of construction and tunnel work for George Jackson, Inc., New York and Chicago.

Mr. J. V. W. Reynders, M. Am. Soc. C. E., Vice-President and Director of the Pennsylvania Steel Co., on Mar. 7 resigned as General Manager of the Steelton, Penn., plant of that company. At the same time he announced the appointment of Mr. Quincy Bent, now Assistant to the President of the Maryland Steel Co. at Sparrows Point, Md., as General Manager of the Steelton plant. The Pennsylvania Steel Co. has been recently acquired by the Bethlehem Steel Co. interests. Mr. Reynders is a graduate of Rensselaer Polytechnic Institute and entered the employ of the Pennsylvania Steel Co. in 1892 as engineer in charge of the bridge and construction department. He was made Superintendent of this department in 1896, remained there until 1906, when he was made Vice-President, a Director and placed in complete charge of the works.

Daniel W. Mead, M. Am. Soc. C. E., and F. W. Scheidenhelm, M. Am. Soc. C. E., have announced their association in the practice of engineering, with offices in the Equitable Building, New York City. They will give their attention to reports, designs and construction covering hydraulic and electric developments, water-supply, flood prevention and reclamation works. Mr. Mead is a consulting engineer, professor of hydraulic and sanitary engineering at the University of Wisconsin and for many years has been engaged in the design and construction of water-works, power plants, dams, etc. He has recently returned from China, where he was one of the three consulting engineers engaged by the American Red Cross to investigate the flood conditions of the Yalu River. He is also consulting engineer to the Miami Conservancy District at Dayton, Ohio. His books on hydraulic engineering are widely and favorably known. Mr. Scheidenhelm has been located in Pittsburgh in consulting practice, subsequent to being in charge of the Cheat River developments of the Hydro-Electric Co. of West Virginia. His practice has been mainly along hydraulic lines, with particular reference to hydro-electric developments and dam construction. For the present, at least, Mr. Mead will divide his time between Madison, Wis., and New York, while Mr. Scheidenhelm will be permanently located in New York.

OBITUARY

Gen. William Sooy Smith died of pneumonia on Saturday, Mar. 4, at Medford, Ore., where he had resided since 1910. He was in his 85th year. A longer biography of General Smith will appear in a coming issue.

F. E. Brandis, a manufacturer of surveying instruments, died at his home in Brooklyn, N. Y., on Feb. 26, at the age of 70 years. Mr. Brandis immigranted to this country from Hanover when he was 12 years old. Scientific instruments were his life study, and he established himself in that business in Brooklyn in 1872. The firm—F. E. Brandis & Sons—is being carried on by the two sons who survive him.

William W. Mills, for many years in the employ of the engineering department of New York City and later a member of the Los Angeles, Calif., engineering corps, died at Las Encinas, Lamanda Park, Calif., recently. Mr. Mills was born in New York City and was a graduate of Rensselaer Polytechnic Institute. He came to New York several years ago to engage in subway engineering work, but when he became ill some months ago, returned to his home in California.

ENGINEERING SOCIETIES

COMING MEETINGS

NEW ENGLAND RAILROAD CLUB.
Mar. 14. Annual meeting in Boston. Secy., W. E. Cade, 683 Atlantic Ave., Boston.

WISCONSIN ELECTRICAL ASSOCIATION.
Mar. 16-17. Annual convention in Milwaukee. Secy., George Allison, Milwaukee.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.
Mar. 21-23. Annual meeting in Chicago. Secy., E. H. Fritch, 900 So. Michigan Ave., Chicago.

NATIONAL RAILWAY APPLIANCES ASSOCIATION.
Mar. 21-23. Annual meeting in Chicago. Secy., B. V. Crandall, 1824 Lytton Building, Chicago.

ILLINOIS GAS ASSOCIATION.
Mar. 22-23. Annual meeting in Chicago. Secy., H. H. Clark, 136 So. Harvey Ave., Oak Park, Ill.

Engineering Society of Wisconsin has elected the following officers for 1916: President, J. S. McCullough; vice-president, A. A. Babcock; secretary, Prof. Leonard S. Smith, Madison, Wis.

National Industrial Traffic League will hold a spring meeting in Baltimore, Md., at the Hotel Belvidere, Apr. 6 and 7. The president is G. M. Freer; the secretary, O. F. Bell, 413 Tacoma Building, Chicago, Ill.

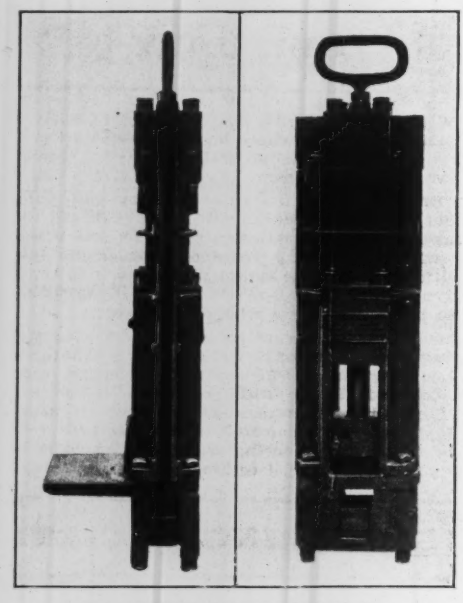
Building Officials Conference, which has been made a permanent organization, as explained in "Engineering News" of Mar. 24, has three classes of membership—active, associate and honorary. Active membership is open to governmental departments that administer building laws and also to individual officials connected with such departments or engaged in the formulation of building laws or ordinances. This membership includes not only state and city building departments, but also labor and tenement-house departments that have to do with the design and construction of buildings and also with members of commissions appointed to draft building laws. Associate membership is open to persons who have been, but are no longer, eligible to active membership. Honorary members may be elected as the result of distinguished services. Active and associate members are to pay annual dues of $5, which will entitle each member to a copy of the annual proceedings. The first dues are payable Oct. 1, 1916. The secretary is S. J. Williams, Madison, Wis.

Appliances and Materials

Small Steam Hammers for Sheeting

A 150-lb. steam (or air) hammer, for driving wood or light steel sheet piling in trenches, that is easily handled by a couple of workmen, has been developed by the National Hoisting Engine Co., Harrison, N. J. This hammer, designated as No. 7, is shown in the accompanying views. It was designed for 2-in. wood sheeting.

The frame has a step, on which the operator may stand to give the extra weight needed for most effective driving. The



LIGHT STEAM PILE DRIVER

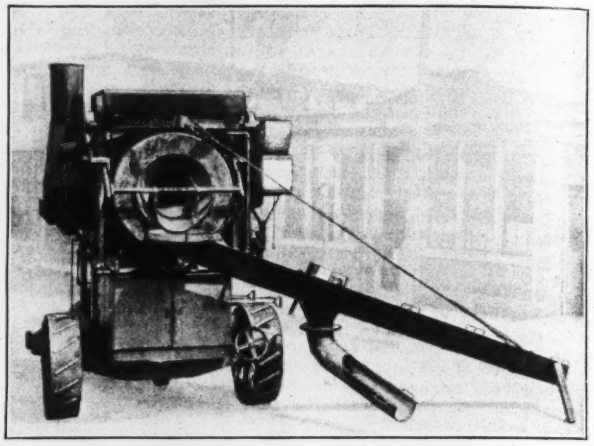
1-in. thickness of overhanging jaw on the striking-block seat allows working in close quarters. The mechanism is very simple, and the hammer can be taken down or assembled in about 15 min. No moving parts project beyond the frame. A simple slide valve on the top of the hammer is thrown back and forth by long trip rods, on each side of the cylinder, worked by lugs on the hammer. The cylinder is 2¼ in. in diameter and has a 4-in. stroke; the piston makes 350 strokes per min. A 5-hp. boiler, or 50 cu.ft. of air per min., is needed to run the hammer.

A similar hammer, designated as No. 6, weighing 650 lb. and intended for 3-in. wood sheeting, is also made by this concern. The cylinder is 3¼ in. in diameter and the stroke 6 in.; 275 strokes per min. is the rated speed. The front striking-block guide plate is only ¾ in. thick for working close up to a ranger.

High-Drum Mixers for Paving

The T. L. Smith Co., of Milwaukee, Wis., showed at the Cement Show, at Chicago, one of a new line of concrete mixers for paving work, known as the "High-Drum Smith Chicago Paver." This type is made in three sizes (with a smaller size of somewhat different details), but they all have the following special features: Wide loading skip, motor traction, chute distribution, automatic water-weighing device, rapid discharge and one-man operation. The accompanying view shows No. 507, and this description applies principally to that model.

The wide loading skip—varying from 7 ft. to 8 ft. 6 in.—permits two or three barrows to be dumped in it at once, thus facilitating the loading. The steam- or gasoline-engine drive, geared both to the mixer and to the tractor, is controlled from the distributing end of the machine, where one man may reach all the operating covers. The chute distributor, shown



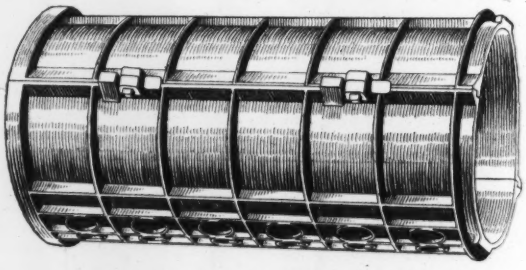
HIGH-DRUM MIXER FOR PAVEMENT WORK

in the view, has a 20° angle of flow, because of the high elevation of the drum, and is provided not only with a swivel at its base, but also with intermediate openings and an auxiliary chute that permit reaching spouting and good distribution of the concrete. The water-weighing device is new. It is automatic in action and can be so adjusted that a given amount of water is regularly emptied into the drum with each batch to be mixed.

* * *

Split Cast-Iron Culverts

Under the trade name of "Panama Culvert" the F. B. Zieg Manufacturing Co., of Fredericktown, Ohio, is making a new split cast-iron culvert pipe. It is claimed that the peculiar construction, shown in the accompanying sketch, provides



SPLIT CAST-IRON CULVERT PIPE

unusual strength and improved fastening for the halves, to make the pipe solid. These pipes may be obtained in 2- and 4-ft. sections of 8- to 36-in. diameters.

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Gasoline Street Lamps

Gasoline street lanterns with double mantle burners for rural districts have been developed by the Coleman Lamp Co., Wichita, Kan. The tank holds 2 qt.—sufficient for 2 hr. burning. The lanterns are intended to be swung from any convenient mast arm with cable and pulley, but bails are provided for portability.